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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

AERIAL TRAMWAYS

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To Be Presented November 2, 1927

Synopsis

The great industrial expansion which has occurred in the United States during the past decade has placed a premium upon the efficiency of machines and labor.

When properly designed and selected, aerial ropeways are very economical and trustworthy machines. The English bibliography is meager. This paper aims at an outline classification, together with such comments and formulas, that may be of service to engineers in determining the elements of an aerial ropeway for their needs.

These formulas have been derived, checked, and used by the writer, and are believed to be in a correct and convenient form.

It is not the object of this paper to discuss the several structures, or equipment, in detail, but as the loading may be determined by the formulas, the design may be developed in the usual way.

Aerial Tramways

Classification.—The terminology used in describing systems of transportation by means of wire rope is loose and indefinite. The following classification of aerial tramways as distinguished from mine haulages, incline planes, or other forms of surface ropeways, will be found in accordance with the views of the most prominent tramway engineers. Abroad, the present tendency is to call the system "aerial ropeways", because of the general application of the word,

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in **March, 1928**.

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"tramway", to surface-haulage installations. Wire-rope aerial transportation systems may be listed as follows:

A.—Single rope tramways (mono-cables):

(*a*) Grip attached to carriers.
(*b*) Clip attached to carriers.

B.—Stationary cable tramways:

I.—Cableways (cable cranes):

(*a*) Radial cableways.
(*b*) Cable hoist conveyors.
(*c*) Dock hoists.
(*d*) Drag-line conveyors.
(*e*) Cable excavators and conveyors.
(*f*) Movable tower cableways.
(*g*) Balanced tower cableways.

II.—Dumping cradleways:

(*a*) Single-span.
(*b*) Multiple-span.

III.—Single cable reversible tramways:

(*a*) Aerial dump.
(*b*) Terminal discharge.

IV.—Double cable reversible tramways:

(*a*) Aerial dump.
(*b*) Terminal discharge.

V.—Double cable tramways with continuous traction rope:

(*a*) Carriers attached by friction grips.
(*b*) Carriers rigidly attached by clips.

VI.—Automatic tramways.

VII.—Bleichert tramways.

VIII.—Bleichert stacking tramways.

IX.—Storage tramways:

(*a*) Circuitous tramways.

X.—Selective turnout tramways.

Rope Tramways

The development of aerial tramways has been rapid and substantial. To Charles Hodgson is given the credit for promoting both the single-rope and double-rope tramways. His original patent was dated 1868. However, the "Hodgson System" is understood to refer to the single-rope type of tramway. This also applies to the "Hallidie System". Tramway lines constructed of this type are rare in North America, but are being installed abroad. They are favored for the transportation of comparatively light materials, such as coffee, tea, tobacco, charcoal, bananas, copra, cotton, etc. However, by means of careful design, they have been used with success in transporting exceptional tonnages (say, 100 tons per hour), of ore, and timber. The economical capacities of such lines are 50 tons per hour, or less. The speed is generally under 450 ft. per min. As the name implies, this type of tramway has a single rope for the simultaneous support and propulsion of the carriers.

Rope.—The rope must be designed so as to meet two conditions: (*a*) The wires must be sufficiently large to withstand the wear and abrasion occasioned in passing over the towers and around the driving sheaves; and (*b*) they must be small enough to be flexible and thus reduce the stresses due to bending. Ropes of six strands and seven wire construction and of crucible grade are satisfactory. Nevertheless, the rapid wear of the ropes constitutes the largest item of the maintenance expenses.

Supports.—Cable supports are either of timber or steel construction, built to 6 or 8-ft. gauge of any reasonable height, and are designed so as to sustain a cap which carries the sheaves supporting the rope. These towers usually consist of a substructure, which is most economically framed with perpendicular posts (Fig. 1), although some manufacturers still favor the batter posts. The tower head is usually standard for any line and can be used interchangeably on any tower. It is desirable that the towers rest on concrete or masonry foundations. However, mudsills or posts set in the ground have been used with success.

Tower Sheaves.—The number of sheaves on either side of the tower may be one, two, three, four, six, or eight, depending on the weight of the load, the radius of curvature desired, and the length of the spans adjoining the tower. When more than one sheave is used, it is customary to carry them in balance by suitable rocker arms. The sheaves are usually 16 to 20 in. in diameter, have shallow rims, rib spokes, and are set-screwed to the shaft, which is carried in babbitted cap bearings.

Carriers.—The load supported by the carriers usually varies from 100 to 1 500 lb., depending on the hourly tonnage to be transported. Loads of 800 lb., or less, are preferable to those of greater weight. If the carrier is equipped with a bucket, the usual volumes are 2 to 10 cu. ft. The bucket is mounted on trunnions supported by swivel castings bolted to U-shaped bails. The trunnions are mounted so that the center of revolution is to one side of the center of gravity. When the bucket latch is tripped, the bucket dumps automatically.

Clips.—A distinguishing feature of single-rope tramways as supplied by different manufacturers is in the method of attaching the hangers to the rope: (*a*) by clips; (*b*) by friction grips. The Hodgson System provided a saddle fitted with India rubber or wooden friction blocks which came in contact with the rope. The friction developed by the weight of the load was sufficient to transport the carriers up moderate grades. Hallidie inserted a clip in the rope. It consisted of a rod with corrugations that were made to conform with the pitch of the rope and the size of the strands. The bucket hanger was forged into a head, which was drilled to receive a pin, so as to permit oscillation in a direction parallel to the travel of the line. The clip was hinged to provide side sway. This method of attaching the carriers to the rope was positive and was successful on lines having steep gradients.

Other types of clips were developed, which depended on placing a thin sheet of metal around and squeezing the rope by means of bolts. Adolf Bleichert and Company developed a friction grip which seizes the rope automatically

when it leaves the loading terminal and is attached the same way at the discharge terminal. This grip is actuated by the weight of the load and is successful on gradients of more than 30 degrees. Manufacturers of that type of tramway on which the carriers are arranged for detaching from the rope, equip the carriages with two wheels so that the carrier may be moved through the terminals when supported on suspended rails.

Automatic Loaders.—Carriers rigidly attached to the rope must be loaded and unloaded while in motion. This arrangement led to the development of automatic loading devices. The loading terminal is arranged to provide a bin for storing the material to be transported. The bin is equipped with a suitable gate so that a container of the same volume as the bucket may be loaded by the operators at will. This, in turn, is discharged into a traveling hopper which is mounted on wheels and runs on a track parallel with the ropeway. The traveling hopper engages the bucket and travels with it a distance sufficient to dump its contents automatically into the bucket. After this is accomplished, it disengages itself and returns to its former position beneath the stationary hopper. This action trips the gate of the stationary hopper and the loader is again filled, thereby completing the cycle. Various aids, such as sprocket wheels, air cylinders, etc., have been used to assist the loader in keeping pace with the bucket without disturbing the latter's equilibrium. When the bucket reaches the discharge terminal, it is automatically tripped and discharged. Carriers equipped with grips do not need automatic loaders or unloaders. They pass to the point most convenient for loading or discharging because of their greater freedom in the terminals.

Angle Stations.—Angle stations may be installed on single-rope tramways to accommodate any change in their alignment. The deflection is accomplished by horizontal sheaves mounted on supporting structures fitted with guides. The slow speed of the tramway permits the carriers to pass around the sheaves either supported directly by the clips or on curved rails, depending on the type of equipment used.

Drive.—The line is driven by either plain, wood-filled, or grip sheaves. The sheaves are fitted with brake rims and friction brakes so that the motion may be properly controlled.

Approximate Cost.—The equipment for a single-rope tramway of 6-ft. gauge, 6-ton hourly capacity, 1 mile in length, provided with an automatic loader, terminal machinery, gears, automatic dumper, supporting sheaves, stands, guides, woodwork bolts and washers, buckets, hangers, clips, crucible steel wire rope, and splicing tools, would weigh approximately 30 000 lb., and would require 28 000 ft. b.m. of timber, including a 20-ton storage bin.

The total cost for such a line installed would be about $18 000. It costs approximately $2 850 per year to operate for 300 days, and would transport 18 000 tons at a net cost of 15.5 cents per ton.

Stationary Cable Tramways

This is the name given to all systems of aerial transportation that sustain the carrier on a stationary cable as a track and propel it by means of

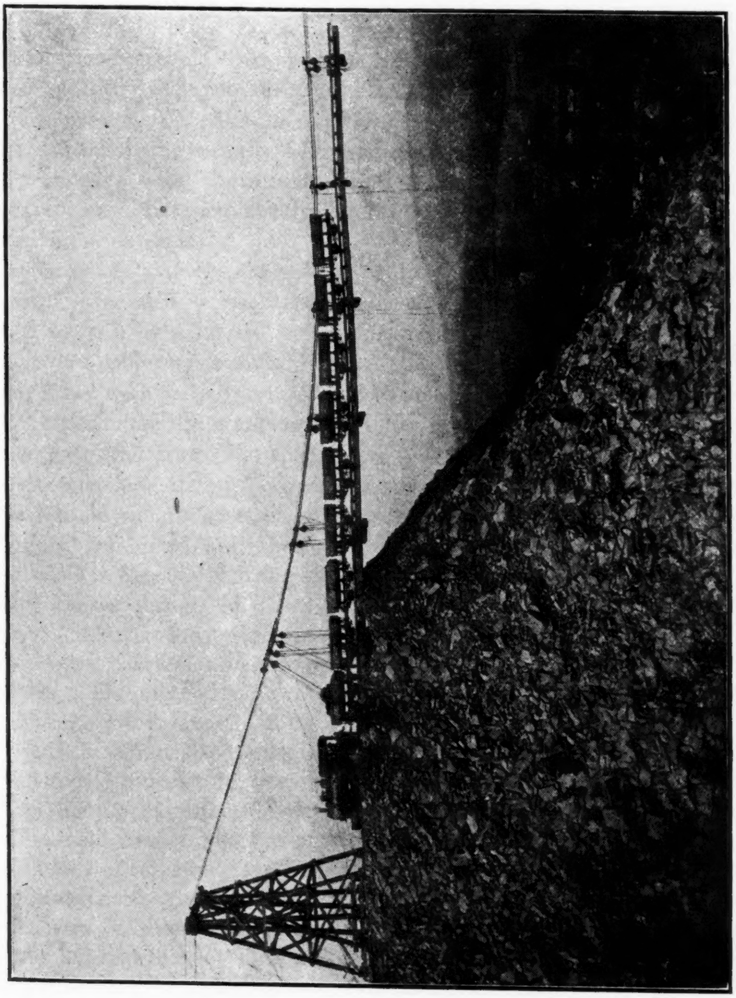

FIG. 2.—DUMPING CRADLEWAYS FOR FILLS.

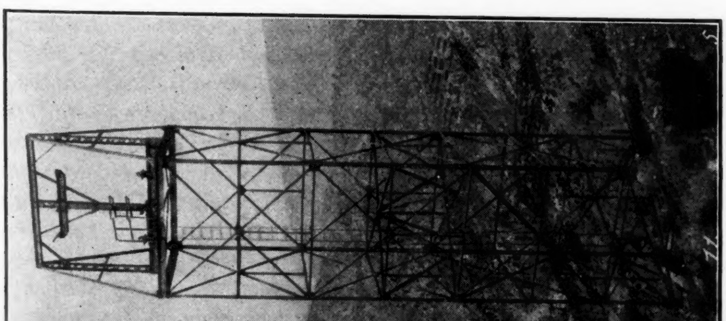

FIG. 1.—STEEL TOWER.

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an independent traction rope. There seems to be a further differentiation among stationary cable tramways, depending on the weight of the unit load handled and the distance through which it is moved. To illustrate, cableways are presumed to handle individual loads weighing from 5 to 10 tons, the translation being usually less than 2 000 ft. On the other hand, double-cable tramways with continuous traction ropes transport loads of 1½ tons, or less to any distance required. All the other classes are intermediate between these extremes.

Cableways (Cable Cranes).—The cableway is defined as a stationary cable tramway which uses a suspended cable for a track and a carriage that is adapted to both hoisting and conveying, in sequence, or simultaneously. The carriage of a cableway is actuated by a reversing engine, either steam or motor-driven, fitted with an elliptic grooved sheave to accommodate the endless rope which is permanently attached to each end of the carriage and is used in moving it to and fro. The engine has a drum for the hoisting rope. If the span exceeds 300 ft., it becomes necessary to support the hoisting rope; otherwise, the weight of the unsupported rope is sufficient to counterbalance the empty fall-block, and the latter cannot be lowered. To overcome this difficulty, devices called fall-rope carriers have been developed. The difference in the design of these parts is the distinguishing feature of the cableways furnished by different companies. Many patents have been issued for fall-rope carriers and several systems have been proposed to eliminate their use. The method now in general use is to support them on a horn of the carriage so arranged as to hold them in place. A stationary rope is fitted with different sized buttons. As the carriage moves, the buttons engage the fall-rope carriers and they are thus automatically spaced along the track cables. When the direction of travel is reversed, the carriage picks them up. To avoid the hammering of the buttons and the wear of the button rope, the differential fall-rope carriers were designed. This design provides that the individual carriers shall move at different speeds, obtained by a change in the ratio of the diameter of the sheaves in contact with the main cable and the hoisting rope. This difference in the rapidity of travel of the differential fall-rope carriers automatically spaces them along the track cable.

When the difference in the terminal elevations exceeds one-third of the span, and when a three-part fall-block is used, the endless haulage rope may be omitted, for the load can be hoisted until the fall-block reaches the carriage before translation will begin. The carriage is returned to position by gravity and is stopped by a block clamped to the track cable. As soon as the motion of the carriage is arrested, the fall-block is lowered to the ground.

Radial cableways are those in which the tail tower is mounted on a circular track. The movement of this tower is usually confined to a sector of 40° about the head tower as a center. When the carrier moves through the head tower to a boom overhanging a boat, the installation is called a "dock hoist". If the fall-block supports a self-filling bucket, such as drag, slip, orange-peel, or clam-shell, the installation is called a "cable excavator and conveyor"; also, a "drag-line excavator and conveyor". Generally speak-

ing, cableways are not adaptable to sites requiring multiple spans, owing to the inability of the usual carriage to pass intermediate supports. Cableways are built with both towers mounted on rails, and, therefore, can be moved laterally, thus commanding a larger area than is possible with the anchored cable. Cableways for storing coal may also be equipped with terminal masts, which swing in a plane at right angles to the cable. As the mast may function as a radius of a circle, the length of back-stays to anchorage is constant for all mast positions. Cableways have been fitted with steel towers resembling a trestle bent which may oscillate about the base. These towers are controlled by weights, which tend to maintain a constant tension in the rope. Cableways are frequently used for the operation of quarries, placer deposits, tailing dumps, etc.

Dumping Cradleways.—When deep fills are made or excessive dumps are to be accumulated at a rate faster than is obtainable by handling single cars, the dumping cradleway provides a method which eliminates trestles, promotes the speed of handling the train, and insures complete salvage of the equipment when the work is finished. Dumping cradleways (Fig. 2) consist of two stationary cables supported by towers located at the end of the span or spans. Suspended from these cables is a cradle which supports a track. The train is pushed to the edge of the fill and the cars are dumped consecutively, the empty cars passing forward on to the deck of the cradleway. The grade of the fill is controlled by a movable tower which is advanced with the cradle as the fill progresses. The elevation of the supporting cables is thereby kept constant irrespective of the position of the cradle on the span. Hold-down tackle keeps the unloaded cradle in position when the cars are drawn from it.

The Cedar Rapids Manufacturing and Power Company, at Cedars, Que., Canada, successfully used a two-span dumping cradleway to build a 2 000-ft. wing-dam of more than 125 000 cu. yd., across a channel of the St. Lawrence River. The cables were 2½-in., 6 by 37, plow steel. The cradle was 14 ft. wide and 130 ft. long, and had a maximum capacity of ten 6-cu. ft. contractors' cars. The height of the movable tower was 40 ft. The cost was reported to be less than $1.25 per cu. yd. The depth exceeded 30 ft., the velocity of current about 10 miles per hour, and a limestone bottom made cribbing and piling methods prohibitive and impracticable.

Single-Cable Reversible Tramways.—This type of equipment differs from cableways in that the hoisting feature is eliminated. Accordingly, the endless traction rope is all that is required. These lines are generally short and of small capacity. The direction of travel of the traction rope is reversed at the completion of each trip of the carrier. Such lines are usually equipped with self-dumping buckets, which may be tripped automatically while moving along the cable, or in stations. The speed of such lines is usually 600 ft. per min., but on clear spans they may be operated as fast as 1 000 ft. per min. The capacity therefore is directly proportional to the load and velocity, but inversely proportional to the span. The hourly capacity rarely exceeds 20 tons.

Table 1 gives the capacities, in pounds per hour, of single-cable, reversible tramways operating at a speed of 600 ft. per min. The time interval to complete the trip is given in seconds.

TABLE 1.—BUCKET CAPACITY.

Length, in feet.	Time, in seconds.	CAPACITY PER HOUR, IN POUNDS.						
		400.	600.	800.	1 200.	1 500.	2 000.	2 500.
2 000	450	3 200	4 800	6 400	9 600	12 000	16 000	20 000
1 500	343	4 200	6 300	8 400	12 600	15 750	21 000	26 250
1 000	240	6 000	9 000	12 000	18 000	22 500	30 000	37 500
500	144	10 000	15 000	20 000	30 000	37 500	50 000	62 500

Double-Cable Reversible Tramways.—For efficiency it is desirable to operate reversible tramways in balance, in so far as the moving equipment is concerned. Most engineers prefer to use two track cables, thus bringing two single reversible tramways side by side, spaced 6, 8, or 10-ft. gauge, depending on the size of the bucket. Both buckets are moved by the same traction rope, which passes around a sheave at the terminal. Tail ropes are required on lines where power is necessary for their operation. Tramways of this type are operated by gravity when there is sufficient fall between the terminal stations. Like single reversibles, they may be equipped with trippers for aerial dumping of the buckets; otherwise, the bucket dumps automatically over the bins in the discharge terminal. This type of tramway is usually arranged so that all its functions are automatic, except the loading and control of the driving mechanism. One operator is sufficient for either single- or double-cable reversible tramways. The track cables are usually anchored without weight-boxes, and multiple spans are used when necessary. The proper determination of the tension of an empty track cable permits the designer to control the ultimate tension of the cable when carrying a maximum load.

The Consolidation Coal Company has used this type of tramway for handling mine waste. At Mine 206, a three-span, double-cable, reversible tramway is installed. It is equipped with 27-cu. ft., bottom-dump buckets which hold approximately 2 000 lb. of slate, and which dump automatically into bins supplying a larry system of distribution. The difference in elevation between the loading and discharge terminals is 427 ft., and the horizontal length is 789 ft. The tramway is operated by a reversing slip-ring motor fitted with a drum controller and resistance-grids capable of 50% speed regulation.

Table 2 gives the capacities, in pounds per hour, of double-cable reversible tramways of different lengths, based on a speed of 600 ft. per min.

Double-Cable Tramways with Continuous Traction Ropes.—Although Messrs. Hodgson and Carrington first considered the arrangement used in double-cable tramways, it remained for Messrs. Adolf Bleichert, J. Pohlig, and T. Otto, of Germany, Cerretti and Tanfini, of Italy, and the Trenton Iron Company, Leschen and Sons, Roebling, The American Steel and Wire Com-

pany, Broderick and Bascomb, Riblett, and others, in America to demonstrate the practicability and perfection of this type of aerial tramway. Two-track cables are used to support the carriers. The usual practice is to increase the diameter of the cable carrying the loaded buckets over that used on the empty side. The traction rope is of the well-known Lang lay construction, is spliced endless, and is controlled by suitable terminal mechanism. The positive attachment of the carrier to the traction rope promotes the development of the highest efficiency possible in any system of transportation.

TABLE 2.—BUCKET CAPACITY.

Length, in feet.	Time, in seconds.	CAPACITY PER HOUR, IN POUNDS.						
		400.	600.	800.	1 200.	1 500.	2 000.	2 500.
2 000	240	6 000	9 000	12 000	18 000	22 500	30 000	37 500
1 500	192	7 500	11 250	15 000	22 500	28 125	37 500	46 875
1 000	144	10 000	15 000	20 000	30 000	37 500	50 000	62 500
500	90	16 000	24 000	32 000	48 000	60 000	80 000	100 000

Tramways built on favorable gradients of 2 to 3% eliminate, owing to gravity, the friction developed by tower rollers, terminal sheaves, carriages, and other moving parts. The power developed or required in the operation of double-cable tramways is then an exact function of the load transported. This is not true for any self-propelled vehicle depending on wheel friction for tractive effort. There is no permanent ratio between the horse power of the prime mover and the load moved. On grades of 12%, the locomotive fails as a tractor under its own weight. Not so with aerial tramways which use, in a modified way, the most efficient system known of transmitting power, for example, by wire rope.

Transportation Costs.—Transportation costs are usually expressed in terms of the ton-mile. This factor is reasonable enough when the haul is of sufficient length for the grades to compensate, thereby presenting costs equivalent to hauling the loads on the level projection of the track. However, with aerial tramways of moderate length—5 miles or less—there is no material increase in operating charges over those experienced on short lines. In other words, the cost of transporting material by aerial tramways is not proportional to the length. The greater part of the expense of operation is for labor in the terminals, so that on equal tonnage lines, one being ½ mile long and the other 2 miles long, the former will show about 400% greater cost per ton-mile than the latter.

Aerial tramways of equal hourly capacity transport the same weight of material per unit of time and the capacity is independent of the length. Hence, generally speaking, the distance of the haul has no effect on aerial tramway transportation costs, as is the case in surface-haulage systems. It is noted that an aerial tramway on a level gradient, and having a capacity of 50 tons per hour, can successfully compete with surface railways less than 17 miles in length. If the topography of the site is rough and broken, double-

cable tramways with continuous traction ropes are supreme. They are especially adapted to the transportation of any substance that can be readily loaded into buckets, 40 cu. ft., or less, in volume, and which produces a net weight of load of 3 500 lb., or less.

Automatic Tramways.—Double-cable tramways with continuous traction ropes, like single-rope tramways, have been developed so that one system attaches the carriers to the traction rope by means of buttons or clips—the so-called automatic system—whereas the other plan is to attach the carriers automatically to the moving traction rope by means of friction grips. The latter type is known as the Bleichert System. The Leschen Special Automatic System is probably the best known type of the button-attached buckets, whereas in the Lawson System, the carriers are attached by clips to the traction rope.

This tends to localize the bending stresses in the traction rope, thereby greatly shortening its life, as compared with lines using friction grips. This latter method is favorable because it is not probable that successive grips will seize continuously the rope at exactly the same points.

The filling of the buckets is accomplished with the aid of an automatic loader, which consists of a hopper having approximately the same volume as the bucket. This hopper is mounted on rails and is movable. It is filled when the operator manipulates the gates of the storage bin. The loader engages a lug on the hanger of the carrier and moves forward in unison with it until the contents have passed to the bucket. The loader then returns to its initial position and, when refilled, completes the cycle of operation. Air pistons, chains, counterweights, and other accessories are used to control the motion of this device. Self-dumping buckets are used, so that when the latch is tripped the bucket revolves on its trunnions, thus discharging its contents. All these operations take place while the carrier is in motion.

In the Lawson or Interstate System parallel track cables are used, which support the carriers, the axes of which are approximately in the horizontal plane. These carriers are dumped when passing over a tail-drum, which reverses their direction of motion, and they return on twin track cables supported on the same structures, but below the cables carrying the loaded buckets. The track cables of both the empty and loaded sizes are sometimes arranged in the same plane, rather than one above the other.

The Leschen System uses carriers hanging in the vertical plane. As a typical automatic double-cable tramway, may be mentioned the "International Line" across the Rio Grande River, which was designed by Leschen and Sons for the Del Carmen Mining Company, of San Antonio, Tex. Its length was 31 500 ft.; its maximum capacity was 10 tons per hour, with a speed of 300 ft. per min. It was equipped with buckets holding 600 lb., which ran on 1-in. and $\frac{3}{4}$-in. track cables of flattened strand construction, cast-steel grade. The traction rope was driven by a 45-h.p. gas engine, which operated a 10-ft. grip sheave. Water carriers used on the line were equipped with hand-attached friction grips, so that they could be removed from the line at will.

Bleichert Tramways.—The Bleichert System and similar systems have carriers equipped with friction grips, which are automatically attached and detached to and from the moving traction rope. The freedom of motion

attained by the carriers in the stations is of prime importance. It permits increasing the speed of these lines to 600 ft. per min. The detaching of the carriers allows their ready control in stations where, of necessity, the rail curves must have short radii. The high speed permits the development of lines of great capacity, such as 275 tons per hour. They have, however, an economical equipment of carriers and track cables. Table 3 gives the elements of Bleichert tramways, in tons per hour; also, the average number of men required on lines of 3 miles, or less. The speed of the line is assumed at 500 ft. per min.

A Bleichert tramway of very large hourly capacity was in service for several years at the mines of the Spring Canyon Coal Company, at Storrs, Utah. It was 3 050 ft. long, with a difference in elevation between the loading and discharge terminals of 330 ft. The capacity was 285 tons per hour. It was equipped with 40-cu. ft. buckets, designed to be transferred from the trucks of the mine haulage system to the hangers of the tramway carriers by a special device called a "tilting table". The empty buckets were placed on the trucks and again entered the mine, thus completing the circuit. The speed of the tramway was 520 ft. per min. Locked-coil steel track cables, in sizes of $1\frac{5}{8}$ in. and $1\frac{1}{4}$ in., were used. The traction rope was of $\frac{7}{8}$-in., Lang lay, crucible cast steel. The towers were equipped with compensating saddles on the loaded side, reducing the bending stresses in the cable. The line was under the continuous control of a sentinel brake. A speed-indicating alarm sounded a warning if the velocity of the line increased 5% above normal. The traction rope passed around an 8-ft. grip sheave equipped with differential, hand-operated brakes. When in operation the line developed 65 h.p. A 75-h.p., 3-phase induction motor could be connected to the drive by means of a friction clutch. The motor was not used except when the line was being stripped of its carriers.

Bleichert Stacking Tramways.—The Bleichert stacking tramways consist of a trussed framework, having bents securely cross-braced and designed so as to be capable of extension by the addition of one or more bents. The tip consists of an independent structure telescoped into the trussed framework. It is mounted on rollers and may be moved outward by screws or hydraulic-jacks. This tip carries a terminal sheave of ample diameter, so that the carriers which ascend the supporting double-headed tramway rail of the framework, pass around it, and are automatically dumped. As the pile grows, the tip is pushed outward. When sufficient space has accumulated, another bent is added to the structure. Piles 250 to 300 ft. in height are not unusual. When the pile has reached a sufficient height, the alignment of the framework is changed to the horizontal plane, and the pile may be extended indefinitely. A sufficient length of travel is provided for the traction-rope tension mechanism so that it is not necessary to splice in additional pieces of traction rope for every extension of the line.

Tramways of this design have been installed at the Brakepan Mines, Limited, of Johannesburg, South Africa, the Boulder Proprietary Gold Mines, Limited, of Kalgoorlie, West Australia, and elsewhere. These installations move as much as 200 tons of waste per hour.

TABLE 3.—BLEICHERT TRAMWAYS, STANDARD EQUIPMENT.

Capacity, in tons per hour.	Size of loaded cable, in inches.	Size of empty cable, in inches.	Size of traction rope, in inches.	Couplings per mile.	Capacity of carriers, in cubic feet.	Spacing of carriers, in feet.	Time interval, in seconds.	Number of carriers per mile.	Number of supports.	Weight, in pounds per 100 ft.*	Cost per 100 ft.*	Weight, in pounds per mile.*	Cost per mile.*	Horse-power required or developed.	Number of chutes.	Terminals, weight, in pounds.†	Terminals, cost.†	Tension station, weight, in pounds.‡	Tension station, cost.‡	Gauge, in feet.	Number of men.
5	1	⅞	½	4	5	1 500	180	9	20	857	$102.13	45 250	$ 5 392	0 to 7	1	26 400	$2 894	7 300	$ 733	6	2
10	1	⅞	½	4	5	750	90	16	20	897	112.49	47 360	5 940	0 to 7	1	26 400	2 894	7 300	733	6	2
15	1	⅞	½	4	5	500	60	23	20	937	122.85	49 460	6 487	0 to 7	1	26 400	2 894	7 300	733	6	3
20	1	⅞	½	4	5	375	45	30	20	977	133.21	51 570	7 034	0 to 7	1	26 400	2 894	7 300	733	6	3
25	1	⅞	½	4	5	300	36	37	20	1 017	143.57	53 680	7 581	0 to 7	1	26 400	2 894	7 800	733	6	3
25	1⅛	⅞	½	4	6	360	43.2	31	22	1 091	146.41	57 610	7 730	7 to 15	2	32 700	3 540	7 800	766	8	3
30	1⅛	⅞	½	4	6	300	36	37	22	1 127	155.53	59 480	8 212	7 to 15	2	32 700	3 540	7 800	766	8	3
40	1⅛	⅞	½	4	6	225	27	48	22	1 191	172.25	62 910	9 095	7 to 15	2	32 700	3 540	7 800	766	8	4
50	1⅛	⅞	½	4	6	180	21.6	59	22	1 256	188.97	66 330	9 978	7 to 15	2	32 700	3 540	7 800	766	8	4
50	1¼	⅞	⅝	5	12	360	43.2	32	22	1 384	183.56	73 070	9 692	15 to 30	3	38 900	4 154	9 100	900	8	4
100	1¼	⅞	⅝	5	12	180	21.6	60	22	1 625	243.48	85 780	12 856	15 to 30	3	38 900	4 154	9 100	900	8	7
100	1⅜	⅞	⅝	6	15	225	27	49	22	1 703	240.27	89 900	12 686	30 to 55	4	59 000	6 087	10 100	1 005	8	7
150	1⅜	⅞	⅝	6	15	150	18	71	22	1 923	292.19	101 520	15 428	30 to 55	4	59 000	6 087	10 100	1 005	8	8
150	1½	1	¾	6	20	200	24	55	24	2 057	290.33	108 590	15 330	55 to 85	5	64 100	6 655	10 400	1 060	8	8
200	1½	1	¾	6	20	150	18	72	24	2 247	333.51	118 640	17 610	55 to 85	5	64 100	6 655	10 400	1 060	8	9
200	1⅝	1	⅞	7	25	187½	22½	59	26	2 390	340.85	126 210	17 997	85 to 110	6	76 300	8 073	12 200	1 250	8	9
250	1⅝	1	⅞	7	25	150	18	72	26	2 554	378.03	134 860	19 960	85 to 110	6	76 300	8 073	12 200	1 250	8	10

NOTES.—Weight of ore taken as 100 lb. per cu. ft.; neutral line; lines requiring power, add for automatic reversing prevention ratchet, 455 lb., $176; lines developing power, add for speed-indicating alarm, 130 lb., $120; power-operated instead of hand-operated chutes, add for each, 2 550 lb., $275; sentinel brake and control mechanism, add 6 200 lb., $850.

* Locked-coil track cables; Lang lay, crucible cast-steel traction rope, standard carriers, equipped with grips, tower saddles, rollers and bolts, telephones and line equipment; that is, all parts affected by change in length.

† Terminal equipment: Sheaves, shafts, pulleys, clutch, tension mechanism, rails, hoods, attachers, detachers, chutes, spacing gong, and tools; that is, all parts not affected by change in length.

‡ Tension station mechanism: Rail, sheaves, rollers, chain, bolts, hoods, etc. Add one station for every mile of line.

Storage Tramways.—These are double-cable, continuous, traction-rope lines equipped with trippers attached to the track cables to trip the latches of the self-dumping buckets. The discharged material accumulates in a pile beneath the tripper. When the pile has reached a height sufficient to interfere with the dumping of the buckets, the tripper is moved farther along the cable. This permits of the accumulation of a pile of relatively narrow width, but of considerable length, in a direction parallel to the tramway. Since the natural slope of such piles is rarely greater than 34°, it is found desirable to increase the gauge of the tramway so as to cover square or circular plots rather than rectangular spaces of great length. This demand has resulted in the development of the circuitous type of storage tramway. Installations of this kind may be provided with overhead grip carriages, or underhung grips, which are capable of passing around curves of 6-ft. radius without being detached from the traction rope. When 12-ft. guide sheaves are used, the speed of the tramway should not exceed 350 ft. per min.; with sheaves, 20 ft. in diameter, 450 ft. per min. The angular throw of the bucket due to centrifugal force can be readily controlled by curved guides when the velocity is within the limits mentioned. Tramways of this type, therefore, consist of towers supporting single-track cables and the traction rope as well as angle stations, equipped with guide sheaves of large diameter. The line follows the perimeter of figures which approach the square or rhomboid in shape. The carriers are always equipped with self-dumping buckets, and it is preferable to use those having the self-righting feature as well.

Bucket trippers may be small pendant clamps, attached to the track cables, or they may be mounted in tripping frames. Tripping frames are of two classes; those that are positively attached to the cable, and, like the clips, can only be moved by sending a member of the tramway crew out on the span to loosen the frame's attachment to the cable. This procedure is more or less dangerous, and, in addition, causes considerable delay in shifting the tripper. The ambulating tripping frame consists of a hood supported on the track cables by rollers. The frame is held in position by four ground lines fitted with proper tackle. The selective ambulating tripping frame carries a toothed bar so that it may be adjusted at will by a line reaching to the ground. The buckets used on tramways equipped with selective ambulating tripping frames are provided with adjustable latch-bars which can be set in the loading terminal according to the material in the bucket. This arrangement permits the handling of various ores simultaneously and storing them in their separate piles or combining them in any proportion desired.

The Tacoma Smelting and Refining Company, of Tacoma, Wash., had such a storage tramway. This line handled zinc and copper ore, or other classes as received, and stored them simultaneously in separate or in bed piles, as the metallurgists decided. The capacity of the line was 100 tons per hour and its length, 3 000 ft. The traction rope was $\frac{5}{8}$-in. Lang lay. The track cables were $1\frac{5}{8}$-in. special locked-coil. The line was equipped with 16-cu-ft. buckets, selective ambulating tripping frames, and pneumatic power chutes. A 25-h.p. vertical motor equipped with drum controller and resistance grids for 50% speed regulation, was used.

Selective Turn-Out Tramways.—This tramway is used where the material transported is to be delivered at several different points, and for the sake of economy the labor required to throw the switches and detachers is eliminated by the substitution of automatic machinery. The carriers are fitted with a shoe attached to a selector bar which has as many notches as there are turn-out stations. The buckets are filled at the loading terminal in the usual way, and the operator sets the selector bar in the notch representing the desired turn-out station.

When the carrier arrives at this point, the shoe engages the throttle of an air-valve, which actuates the pistons of two cylinders controlling the swinging detacher and the turn-out switch. The carrier automatically releases the traction rope, enters the siding, and coasts to the proper point under its own momentum. As it leaves the station, the shoe encounters a release valve, which restores the mechanism to its original position. Any carrier with the selector bar set for another station passes by this switch, because the shoe on its carriage does not engage the air mechanism.

Economic Design.—The structures of an aerial tramway are grouped into the terminal stations, which are usually independent of the length of the line and, for a given tonnage, represent a fixed cost. Tension and other intermediate structures are required only occasionally along the tramway and may be also regarded as additional cost factors. Towers for any given line will be of standard design, and, therefore, the criterion to apply in choosing the capacity of the carriers and the size of cable for economy of design is to select them so that the cost per foot of cable multiplied by the bucket spacing plus the cost of a carrier shall be a minimum, or as near thereto as the time interval will permit.

Track Cables.—Track cables are an essential part of all systems of aerial tramways. Four styles of cable have been used for this purpose. They are locked wire, locked-coil, smooth-coil track strand, and patent flattened strand. The first three are shown in Fig. 3.

Locked wire cable is so named from the fact that the outer wires have a Z-shape so that they interlock one with the other and thus present a smooth surface on the exterior, resulting in a uniform distribution of wear of the wires, which cannot be attained by the use of any other cable. Experience has shown that no other track cable gives as great a tonnage life as locked wire and locked-coil cable. Table 4 gives the properties of locked wire cables.

Locked-coil track cable is very similar to locked wire cable, except that the core and key wires are of larger cross-section, so that the cost of manufacture is reduced and likewise the price to the trade. Fig. 4 shows a test specimen of this cable. (Note the holding power of the zinc sockets.) It is satisfactory for use as a track cable for aerial tramways carrying net loads of 5 000 lb., or less, whereas locked wire cable is greatly favored for use on cableways where the weight of the moving load is approximately 8 to 10 tons. Table 5 gives the characteristics of locked-coil cable.

TABLE 4.—Properties of Locked Wire Cable.

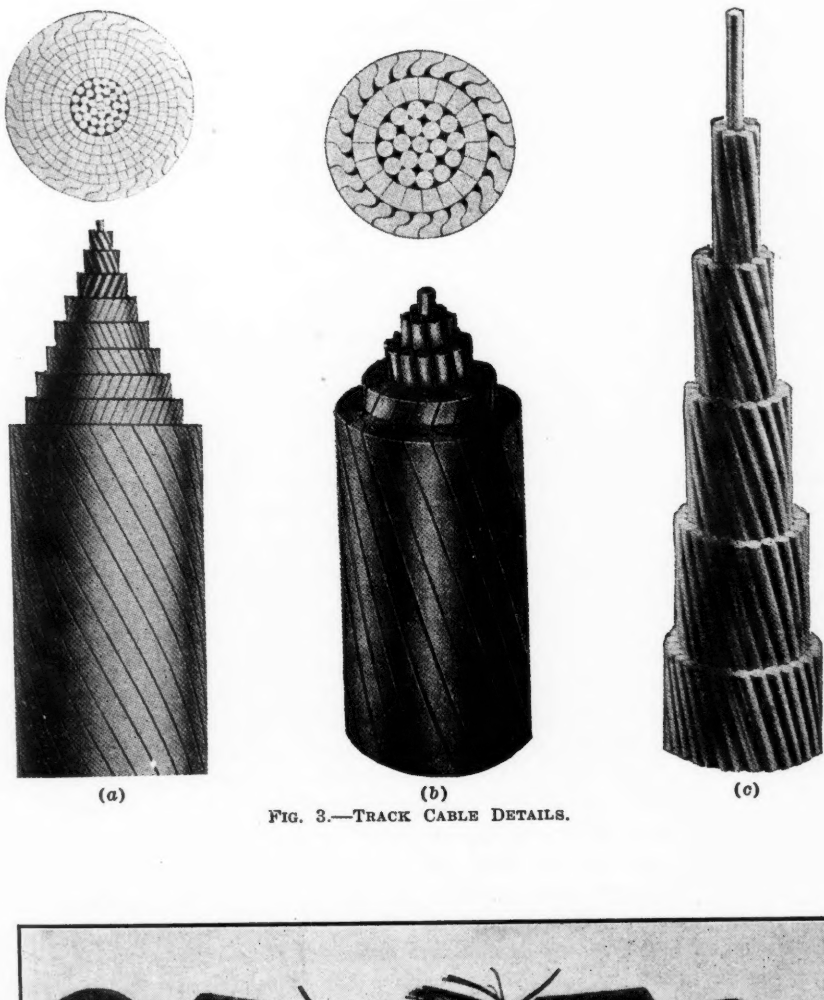
Diameter, in inches.	Area, in square inches.	Approximate weight per foot, in pounds.	Approximate breaking stress, in tons.
2¼	3.410	12.50	190
2	2.725	10.00	160
1¾	2.080	7.65	120
1⅝	1.800	6.60	103
1½	1.550	5.70	89
1⅜	1.290	4.75	75
1¼	0.040	3.80	62
1⅛	0.858	3.15	50
1	0.680	2.50	40

When a cheaper track cable is desired, smooth-coil track strand is used. This cable is composed of a number of comparatively large round wires laid about a core wire. The number of wires and their size vary according to the size of the cable, 19, 37, and 55 being the usual construction for cables of the size used on most tramways. Should a wire break, it uncoils rapidly to the points of support of the span. With locked-coil cable, it is necessary for two or more adjacent wires to break before the wires can run. Smooth-coil track strand is superior to all other track cables except the locked wire and locked-coil types. The use of cables, such as patent flattened strand, or of the ordinary type of wire ropes, on permanent installations, is not good practice, for the life of such cables is frequently extremely short, due to the breakage of the small wires which must be used in their construction. Table 6 gives the characteristics of smooth-coil track strand.

TABLE 5.—Locked-Coil Cable.

Diameter, in inches.	Area, in square inches.	Approximate weight per foot, in pounds.	Approximate breaking stress, in tons.
2	2.60	9.4	160
1⅞	2.30	8.2	140
1¾	2.05	7.30	120
1⅝	1.80	6.30	103
1½	1.55	5.30	89
1⅜	1.35	4.40	75
1¼	1.10	3.70	62
1⅛	0.85	3.00	50
1	0.70	2 85	40
⅞	0.50	1.80	30

The modulus of elasticity of track cables in both bending and direct tension varies with the stress imposed. When a load, such as a moving carrier, passes along the track cable, bending stresses are developed in the cable. The relationship between the weight of the loaded carrier and the size of the track cable is based on the criterion that the combined bending and direct stresses in the outermost wires must be a minimum. The weight of a carrier that may be safely sustained by a track cable varies with the tension between limits of 300 to 700 times the weight per foot of the cable.



(a) (b) (c)

FIG. 3.—TRACK CABLE DETAILS.

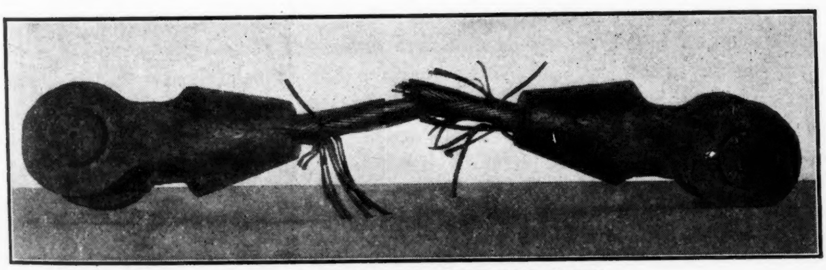


FIG. 4.—TEST PIECE OF LOCK-COIL CABLE.

TABLE 6.—SMOOTH-COIL TRACK STRAND.

Diameter, in inches.	Number of wires in strand.	Area, in square inches.	Weight per foot, in pounds.	Breaking Stress, in Tons.	
				Crucible steel.	Plow steel.
2	61	2.24	8.40	185.00	218.00
1⅞	61	1.97	7.28	161.00	189.00
1¾	61	1.72	6.59	145.80	171.00
1⅝	61	1.49	5.63	124.00	146.00
1½	37	1.27	4.88	108.40	127.50
1⅜	37	1.06	4.01	88.80	105.00
1¼	37	0.86	3.23	71.80	84.60
1⅛	37	0.72	2.70	60.00	70.70
1	19	0.56	2.20	49.20	58.00
⅞	19	0.44	1.69	37.60	44.40
¾	19	0.32	1.24	27.60	32.50

Tramway track cables show, when properly tested, several interesting phenomena which explain their behavior in service. The elongation of a specimen of locked-coil cable is not the same for equal values of progressive and retrogressive loading. If elongations are plotted as ordinates and the loads as abscissas, a frictional hysteresis graph is developed, and is capable of being interpreted in the same manner as the well-known engine indicator diagrams. The area bounded represents work. As the rate of application of the loading is very slow, the heat generated is small. Hence, the work shown is a measure of the internal friction developed among the wires. This force is comparable with the effect of cohesion in homogeneous bodies. As the internal friction is a function of the tension, so is the modulus of elasticity. The cable specimen does not follow Hooke's law until the tension is well advanced. Because of these properties the tonnage life of track cables is profoundly influenced by the manner of their support, the weight of the moving carrier, the amount and the method of applying the tension, and the lubrication and care received.

Radius of Saddles.—In aerial tramway designing, the ratio of load to tension should be taken as less than 0.05. As the wheel base of the carriage is known, the radius of curvature of the cable between the carriage wheels can be approximated. This value, or a greater one, should be used for the radius of the track cable saddles, so that the bending stresses in the cable at the supports will be of the same order as those developed under the carriage. If the wheel base of the carriage is 1.2 ft., the radius of curvature of the cable should be about 25 ft. Fig. 5 shows a rocking saddle.

Anchored Spans.—As track cables under high tension show properties similar to homogeneous bars, it is well to bear in mind that heavy boxes promote the development of great internal friction; in other words, there is no relief to the constituent wires from the stresses imposed by the passing carriers. These stresses are not reduced by stopping the tramway. This continuous loading results in reduced tonnage service and engineers have given their attention to the merits possessed by anchored spans.

If a loaded carrier traverses a cable span, the ends of which are firmly anchored, and the cable is assumed to be non-elastic, the only stress effect

observed is an increase in the tension as the carrier moves from the support toward the center; when passed, the tension declines to that of the empty span as the carrier passes the support. As the erection tension of the empty cable is less than that required to develop an internal friction sufficient for the cable to show an elongation proportional to the stress, it follows that the continual change in the tension due to the passing carrier results in a mutual adjustment of the wires with regard to the applied stresses. The result is that broken wires are less frequent, and the usefulness of the cable is enhanced. The objection of tramway designers to the use of anchored spans is due to the difficulty of computing the erection tension of the cable so as not to exceed, when loaded, a pre-determined amount. By the use of formulas developed from the principles of the elastic theory, this objection has been overcome.

Lubrication.—In order to keep the internal friction at a minimum and protect cables from the elements, the aerial tramway track cables and traction ropes should be thoroughly lubricated with a non-acid oil. Tramway manufacturers can supply track-cable oilers which are equipped with tanks and oil pumps (some are operated by compressed air) belted to the carriage wheels and, when they are in motion, the pumps deliver a constant stream of oil to the cable. Traction rope oilers are maintained in the terminals, and consist of tank, sheave, and wiping brushes. The sheave runs in oil and thus lubricates the rope. To insure uniform wear on the exterior surface of a track cable, experience has shown that it may be turned so as to present a different surface to the carriage wheels. If cables are turned, the operation must be done methodically. It will not do to operate a cable for a long time in one position and thereby allow the wires to assume a permanent set, due to the development of large internal adhesive forces, and then turn it, for the cable will then show broken wires very quickly. If a tramway has a capacity of 50 tons per hour and the cables are given a one-quarter turn for every 12 000 tons transported, and the other conditions of support, tension, and lubrication are favorable, the tonnage life will be increased. It has been recorded that more than 2 000 000 tons traversed a $1\frac{1}{4}$-in. locked-coil cable before complete replacement. Large sized track cables have not as yet developed a tonnage life equal to that secured from small cables, but by using the proper principles of design, the tonnage life is being greatly augmented.

Couplings.—Locked-coil and smooth-coil track cables cannot be spliced in the usual acceptance of the term. They are joined by a special device called a coupling, which consists of three pieces: Two sockets of nickel steel which are joined by a central plug having right and left-hand threads; the unscrewing of the several parts being prevented by dowel pins. The ends of the cable are introduced into the tapered sockets and are flared by thimbles driven between the several layers of core, key, and lock wires. The wires are gathered into groups by means of wedges driven lightly between them. Fig. 6 shows three lock-coil cable couplings that proved to be 92%, 80%, and 60%, respectively, as strong as the cable. The diagram also shows the nature of the failures resulting from improper driving of the thimbles.

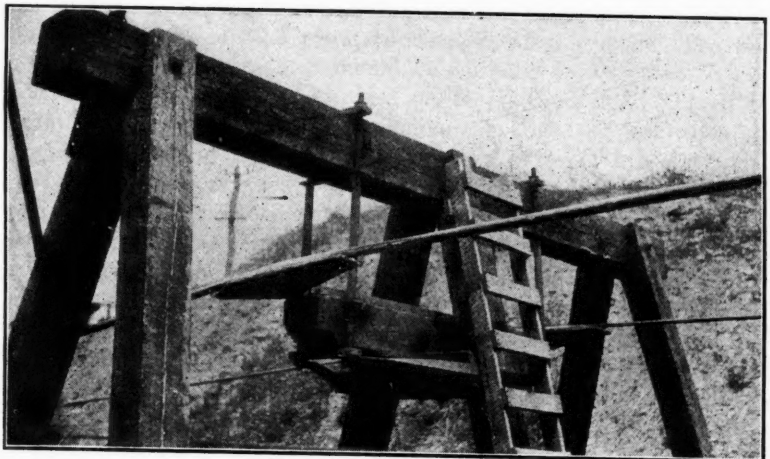


FIG. 5.—ROCKING SADDLES.

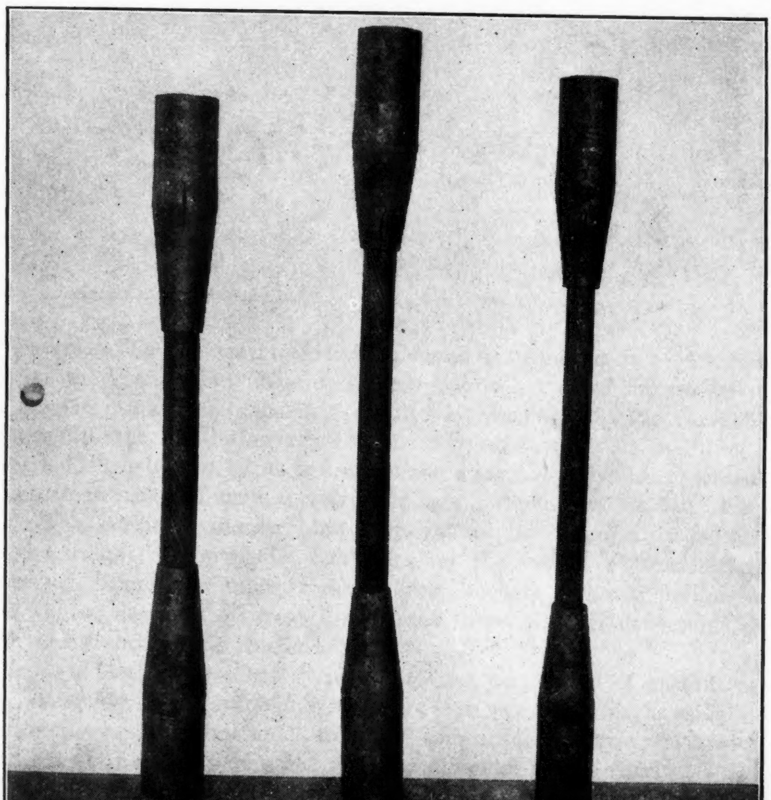


FIG. 6.—LOCK-COIL CABLE COUPLINGS.

This system of joining cables is very satisfactory, as the coupling frequently develops 90% of the ultimate strength of the cable. The sockets are made from nickel steel, and should be specially heat-treated so as best to stand the bursting pressure imposed on the tapered shell. Track cables do not fail simultaneously at all points along the line. Broken wires appear and are either welded by the oxy-acetylene blow-pipe torch, or a section of the cable is removed and a new piece coupled in, which is a very simple operation.

Traction Rope.—Practically all double-cable tramways are equipped with traction ropes made of six strands of seven wires coiled about a hemp center in a manner known as Lang lay. Steel of a grade higher than extra strong crucible is seldom used because of the hardness of the wires and the consequent tendency to rupture. Lang lay rope has the wires approximately parallel in adjoining strands, which increases the flexibility and gives a greater surface of wear. Traction ropes should be lubricated regularly if satisfactory service is to be obtained. Table 7 gives the properties of Lang lay traction ropes.

TABLE 7.—CRUCIBLE STEEL TRAMWAY TRACTION ROPE, LANG LAY.

Diameter, in inches.	Area, in square inches.	Approximate weight per foot, in pounds.	Approximate strength, in tons.	Proper working load, in tons.
1 1/8	0.4606	2.	37.	7.4
1	0.3822	1.58	31.	6.2
7/8	0.2884	1.20	24.	5.3
3/4	0.2129	0.89	18.6	4.7
11/16	0.1760	0.75	15.4	3.8
5/8	0.1565	0.62	13.	3.1
9/16	0.1217	0.50	10.	2.4
1/2	0.0954	0.39	7.7	2.

Carriers.—The ordinary carrier in general use consists of a carriage that traverses the track cable. The carriage supports a trussed hanger by means of a hanger pin, which slips into a suitable receptacle in the carriage and thus permits the oscillation required by various grades on the line. The hanger holds the bucket by its trunnions and a latch. It is also provided with the necessary parts to sustain a friction grip, which may be one of several types. The carriage, hanger, and grip should be designed in accordance with the weight of the load. Table 8 gives the weight of standard empty carriers of different volume. Self-dumping buckets and odd designs weigh more. These data are of great importance in calculating deflections, horsepower, and traction-rope tensions.

Grips.—Friction grips are divided into two classes: (1) Constant closure grips, arranged so that the movement of the jaw is fixed, as long as the adjustments remain constant; and (2) compensating grips, in which the movement of the jaw is dependent on the diameter of the rope.

Grips are further sub-divided as to style, into side-opening, top-opening, and bottom-opening, depending on whether the traction rope enters the grip from the side, top, or from below.

TABLE 8.—SIZE AND WEIGHT OF STANDARD EMPTY CARRIERS.

Size of bucket, in cubic feet.	Weight of carrier, in pounds.	Size of bucket, in cubic feet.	Weight of carrier, in pounds.
5	315	20	560
6	330	25	640
8	370	30	700
10	435	35	775
12	455	40	830
15	480		

Constant closure grips are satisfactory because the position of the closing lever is under perfect control. They are of great utility on lines having only one section of traction rope. The compensating grips, on the other hand, should be used on multi-section lines, because traction ropes, irrespective of equal stress, do not wear so as to maintain equal diameters. The Webber grip is an example of the constant closure side-opening type, the closure of the jaws being self-locking under a positive pressure, the amount depending on the position of the adjusting screws. This is classed as an underhung grip, meaning that it is mounted on the hanger below the track cable. In traversing long spans, under conditions where the traction rope has a severe tension, underhung grips show a tendency to lift from the cable. The Bleichert grip may be either overhead or underhung and is one of the best known top-opening grips. It depends on the weight of the load acting through proper leverage on the grip jaw for its holding power. It will easily handle ropes of $\frac{3}{16}$-in. difference in diameter. The Pohlig grip is of simple design and has a bolt carrying a coarse and fine thread. The coarse thread insures rapid movement of the jaws for a limited throw of the closing lever. The finer thread acts as a differential, imposing great pressure on the rope for the final movement of the closing lever. The position of the lever, when the grip is closed, is not constant, but varies with the wear of the parts and the traction rope. It, therefore, requires constant adjustment to prevent serious accident due to the closing lever under-running the detacher. The "Wico" compensating grip is regarded by many as the best friction grip yet developed. All styles of grips for satisfactory service should develop a resistance to slipping of 1 500 lb., or more, when properly closed on the traction rope.

Buckets.—Manufacturers of tramways will supply any style of bucket desired, such as self-dumping, self-dumping and self-righting, bottom dump, etc. When buckets are not required, special receptacles are used, depending on the nature of the material to be transported.

Supports.—These may be of wood or steel and cover three classes of structures, namely, towers, rail structures, and bent supports. Towers are of two general types (*a*) those in which the track cables may be mounted without threading them through the tower; and (*b*) those with closed heads. Towers of the first type are very satisfactory for loads up to 1 500 lb. Those of the second class are battered on a slope of 1 to 10 in a direction at right angles to the center line of the tramway. Tramways are usually designed

with 6, 8, or 10-ft. gauges, depending on the size of the receptacles, transition being made between 6 and 8-cu. ft. buckets for the 6-ft. gauge, and 30 to 35-cu. ft. buckets for the 8 and 10-ft. gauges, respectively. Table 9 gives the quantities of timber in both types of supports of various heights for both 6 and 8-ft. gauges.

TABLE 9.—APPROXIMATE TIMBER CONTENT OF TOWERS, IN BOARD FEET.

Type.	Height, in Feet.								
	10	12	15	18	20	22	25	30	35
Closed type, 6-ft. gauge....	685	850	970	1 080	1 190	1 280	1 370	1 640	1 790
Open type, 6-ft. gauge......	455	455	780	860	918	996	1 073	1 338	1 502
	Height, in Feet.								
	40	45	50	55	60	65	70	75	80
Closed type, 6-ft. gauge....	1 950	2 265	2 440	3 100	3 275	3 780	3 930	4 350	....
Open type, 6-ft. gauge......	1 887	2 087	2 410	2 568	2 883	3 041	3 325	3 678	3 887
	Height, in Feet.								
	10	12	15	18	20	22	25	30	35
Closed type, 8-ft. gauge....	916	1 180	1 340	1 770	1 940	2 241	2 385	3 021	3 254
Open type, 8-ft. gauge......	....	....	886	906	974	1 047	1 119	1 423	1 547
	Height, in Feet.								
	40	45	50	55	60	65	70	75	80
Closed type, 8-ft. gauge....	4 016	4 244	4 980	5 470	6 000	7 180	8 360	9 180	10 000
Open type, 8-ft. gauge......	1 967	2 098	2 458	2 572	3 002	3 143	3 410	3 765	3 989

Table 10 gives the approximate weight of steel supports, depending on the height and gauge.

TABLE 10.—MINIMUM WEIGHT OF STEEL TOWERS.*

Height, in feet.	6-ft. gauge, in pounds.	8-ft. gauge, in pounds.	Height, in feet.	6-ft. gauge, in pounds.	8-ft. gauge, in pounds.
10	1 720	2 140	35	4 850	5 140
12	1 860	2 310	40	5 400	6 030
15	2 240	2 880	45	5 900	6 620
18	2 650	3 130	50	6 600	7 860
20	3 100	3 500	55	.....	8 000
22	3 280	3 680	60	.....	9 100
25	3 550	3 900	65	.....	9 910
30	4 180	4 690	70	.....	10 400

* For permanent construction, increase these weights 50 per cent.

When the tramway survey crosses a crest or summit of such height that a vertical curve of small radius is developed, tramway engineers specify rail structures composed of successive bents that support the carriers on tramway rails overhanging the cables, and thus accommodate the required curvature. These structures relieve the track cables of the severe stresses due to bending.

When the rate of change of the vertical curve for a chord length (determined by the spacing of the bents) is sufficiently moderate so that the cable can be used for the support of the carriers, the structure is called a bent-support. It is seldom that these structures have more than four bents, and they are not as desirable as rail structures when the slight difference in cost is neglected.

Tension Structures.—On tramway lines exceeding 1 mile in length, it is frequently necessary to use tension structures for the purpose of supporting weight-boxes which are connected to the track cables. This obviates the necessity of attempting to maintain a uniform tension in a cable passing over a great number of tower saddles. Tension structures are equipped with tramway rails so that the carriers pass from one section of cable to the next without difficulty. The traction rope is continuous and is supported by rollers. If the cables of one section are anchored and the other section is controlled by weights, the structure is known as an anchorage and tension structure. There are also double anchorage and double tension structures, depending on whether the four cables are anchored or weighted.

Guard Nets and Bridges.—In crossing public highways or railroads, it is sometimes necessary to provide protection against the premature discharge of the bucket, or its derailment. Such guards may be steel structures or suspended nets. Accidents, however, are of very rare occurrence, and unless the traffic is considerable, guard nets are installed for safety. Most railroads require an overhead clearance to the under side of the guard screens of more than 22 ft. Fig. 7 illustrates the use of guard screens.

Terminals.—This is the name given to the stations at the ends of the tramway. They are designed and equipped so as to make the operation of the tramway as nearly automatic as possible. On lines of ordinary length, the loading and discharge terminals only are required. In the former, the buckets are loaded (see Fig. 8), and, in the latter, they are discharged. Hence, the names, loading and discharge terminals.

Lines of considerable length, and especially those having a great difference in elevation between the loading and discharge terminals, must be divided into sections so as to relieve the stress in the traction rope. This requires the use of intermediate control stations. Also, if angles are desired, they may be installed at intermediate control stations or by the use of angle towers. The judicious utilization of angle towers frees the designer of aerial tramways from the limitation that they must be built in straight lines between stations.

The terminals are equipped with rails so that the carriers may run freely to the loading and discharging points. In these stations are located the

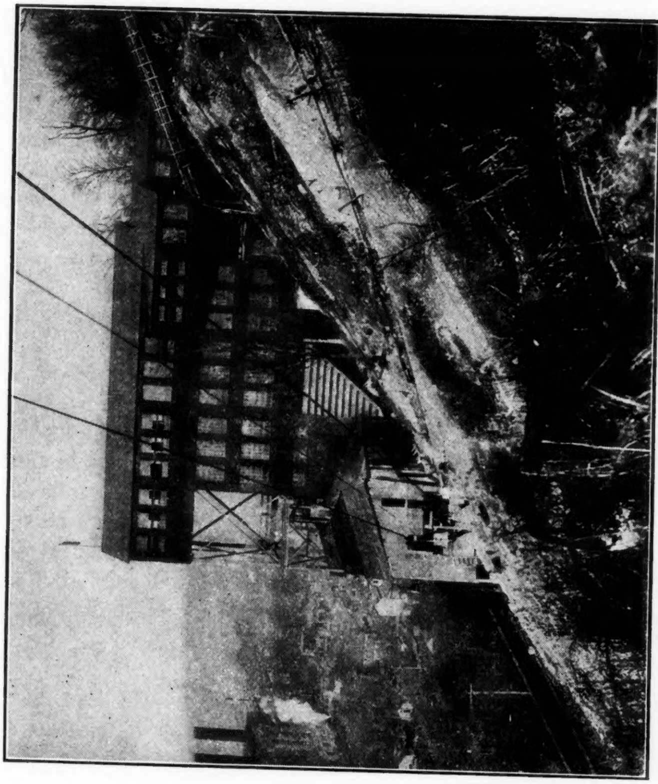


Fig. 8.—Loading Terminal, Mine Waste at Tipple.

Fig. 7.—T-Rails Used for Guard-Screen Flocr.

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attachers and detachers, which automatically close and release the levers of the carrier grips.

When the velocity of the traction rope is 500 ft. per min., or more, angle stations are not desirable. However, when the velocity of the line is 400 ft. per min., or less, angle stations with continuous traction ropes may be used, and the direction of the carrier is changed by passing it around guide sheaves of large diameter. In this case, the carriers are frequently equipped with grips of the overhead type, designed so that the jaws will pass freely between the traction rope and the sheave, irrespective of the direction of travel of the carriers. In properly designed angle stations underhung grips may be used and the large diameter deflating sheaves omitted.

Bins.—As usually installed, aerial tramways load from, and discharge into, bins. Storage facilities at the loading and discharge terminals permit freedom in operation, for production delays do not immediately stop the tramway; and, similarly, the tramway delays do not stop the operation of the plant. As bins of large capacity are built of a number of cells, Table 11 will be found of service in estimating the dimensions of square cross-section bins with bottoms sloping 45 degrees. These dimensions are computed so as to give a maximum volume with a minimum surface for the style of bin shown. The volume is stated in cubic feet and also in tons for material weighing 100 lb. per cu. ft. The dimensions apply to the inside of the bin and the framing should be arranged accordingly.

TABLE 11.

Volume, in cubic feet.	Capacity, in tons per 100-lb. per cu. ft.	Sides of square cross-section, in feet.	Center depth, in feet.	Depth of front face of bin, in feet.	Depth of back face of bin, in feet.
100	5	5.21	3.67	6.28	1.06
200	10	6.57	4.63	7.92	1.34
300	15	7.51	5.29	9.04	1.54
400	20	8.29	5.84	9.99	1.69
500	25	8.92	6.27	10.73	1.81
600	30	9.49	6.67	11.42	1.92
800	40	10.41	7.34	12.55	2.13
1 000	50	11.24	7.92	13.54	2.30
1 500	75	12.86	9.07	15.50	2.64
2 000	100	14.16	9.97	17.05	2.89

Controllers.—Tramways having considerable difference in elevation between the loading and discharge terminals, operate by gravity, and, if not controlled, tend to accelerate until the line is wrecked by "running away". Such installations are equipped with efficient differential hand-brakes and with sentinel brakes. The sentinel brake consists of revolving cone-shaped vanes attached to hinged arms so that the radius of the path of travel changes, and hence the power absorbed varies with the cube of the speed. The sentinel brake is easily adapted to absorbing from 10 to 100 h.p. Properly selected multi-vane fans may be used, also, for tramway control.

It was formerly the custom to use hydraulic controllers, which were designed on the principle of the cycloidal pump. A valve was provided which

controlled the discharge pressure of the fluid and thus varied the resistance of the regulator. Owing to the excessive wear of the moving parts of this device, the high fluid pressure, and the danger of freezing in cold weather, it has been generally abandoned.

Tramways have been designed wherein the buckets served both as bodies of mine cars and receptacles for the aerial tramway, the transfer being effected in the loading terminal by suitable mechanism. This arrangement is not altogether desirable, for the reason that aerial tramways and mine haulages are decidedly different in principles of operation, and the class of equipment which is ideally adapted for use as mine cars is not completely satisfactory for tramway buckets.

Tramways are often equipped with elevators for raising and lowering either the empty or loaded carriers from one level to another. This usually happens when the returning carriers are loaded with back freight, which is discharged on the upper floor commanding a warehouse, and the buckets are then dropped to the lower floor, which is convenient to the loading bin.

Scales should be located in a section of the tramway rail, so that the weight of every carrier can be ascertained. These scales are equipped with type-registering beams or plain beams as required. Counters may also be installed so that the exact number of carriers passing a given point can be ascertained at any time.

Speed-indicating alarms show the velocity of the tramway at any instant. Should it exceed the predetermined speed by 5%, an electric gong sounds, which calls the operator's attention to this dangerous condition, and he may apply the brakes before disaster occurs. Special gongs are used to sound a signal for despatching the carriers, thus insuring their uniform spacing. Electrical dispatching and loading control systems that are available reduce the toil of terminal labor to a minimum.

Reverse preventers, installed on the driving machinery of tramways, would reverse their direction due to the action of gravity on the loaded buckets. This condition always exists when the discharge terminal is at a higher elevation than the loading terminal.

Solenoid brakes have been proposed for use on aerial tramways that are motor-controlled. Alternating current is usually available, especially in the mining district. Induction motors, therefore, are used to drive or brake the tramway. Such motors "run away" when the power circuit is interrupted. The solenoid brakes operate when the circuit is broken. However, care must be utilized in choosing the size of brake so as to prevent wrecking the line due to its sudden stoppage. The speed of tramways having electrical drives is most conveniently varied by the use of drum controllers, resistance grids, and slip-ring type of motors; 50% speed regulation is easily obtained by this method.

Chutes.—A great variety of gates are used to control the flow of material from the loading bins to the tramway buckets.

Tramway Profile.—The tramway profile should be made by a competent surveyor and should have an accuracy of at least 1 to 1 500. The profile should

show the topography 25 ft. each side of the center line joining the loading and discharge terminals, and the plan should show all the control points at the loading and discharge terminals. Particular detail should be given of railroad tracks, intervening structures, the shaft, or adit, as the case may be. The topography on 2-ft. contours should be given of the sites of the loading and discharge terminals, as well as of the intermediate structures. The slope of the ground surface at right angles to the line should be indicated in degrees. It is always understood that the initial station of a tramway survey is located at the loading terminal. Accordingly, slopes may be indicated to the right or left of the line without confusion.

In crossing summits the elevations should be carefully noted, for the reason that the tramway will probably require a rail structure at such points. In crossing valleys where the depth is greater than one-tenth the span, it is not necessary to carry the line to the bottom, but it should be carried well down the slopes. This is particularly true for the ends of spans which have different elevations. If the length of the line is in excess of 5 000 ft., it should be plotted 100 ft. to the inch, the horizontal distances and elevations being drawn to the same scale. On shorter lines, a scale of 40 ft. to the inch is preferable. However, profiles when analyzed mathematically may be drawn to convenient scales, as 1 in. = 500 ft. horizontally, or 1 in. = 200 ft. vertically.

Stadia surveys, carefully made, are satisfactory, but the accuracy of the work is put to the final test when the track cables are stretched from terminal to terminal like a giant tape measure. Errors in elevations appear when the cable fails to rest properly on the saddles of the supports. The position of track cables when empty and loaded can be computed with such accuracy that it is not necessary to raise or lower the towers except when errors exist in the survey. The tramway designer sometimes adds to the purchaser's expense the cost of shifting supports, because of negligence or unwillingness to analyze accurately the characteristics of the cables at the point of support and thus determine the proper elevation of the saddles so as to develop proper cable pressures.

The profile should also state the horizontal length of the proposed line; the difference in elevation of the terminal points; the capacity of the tramway, in tons per hour; the weight of the material per cubic foot when broken to the size that will be delivered to the buckets; and the nature and location of the power; also, all railroad crossings, power lines, roads, buildings, snow, grazing or cultivation clearances, or other obstacles, which the tramway must cross. The minimum clearance between the ridge of a building and the track cable is 10 ft. Most railroad companies specify a minimum clearance of 22 ft. between the bottom of the tramway bucket and the top of rail.

These points should be well considered by the engineer who makes the survey. If the work is carefully done, it is not necessary to supply the tramway manufacturer with field notes other than the data shown on the profile. The elevations and station numbers should be carefully noted.

Tables 12 and 13 are given as aids in approximating the size and timber content of the several structures used on tramways. They show the name of the station, its volume, in cubic feet, the number of feet board measure, the ratio of board feet per unit volume, and the length, width, height, and capacity.

TABLE 12.—TIMBER CONTENTS OF TRAMWAY STRUCTURES.

Name.	Volume, in cubic feet.	Feet board measure.	Feet board measure, volume.	Length, in feet.	Width, in feet,	Height, in feet.	Tons per hour.	Average of
Loading terminal......	18 260	15 000	0.85	67	22	15	37.5	24
Discharge terminal ...	50 500	25 500	0.61	62	24	35	40	24
Anchorage and tension structure.......	13 900	13 500	1.00	40	14	26	21	11
Anchorage, tension and curve rail.......	23 400	19 400	0.83	75	14	24	20	5
Curve rail structure ..	15 250	12 075	0.96	53	14	20	22	19
Rail structure.........	13 450	10 830	0.92	20	14	45	20	6
5 bent structures......	9 785	6 210	0.63	35	14	20	52	2
4 bent structures......	5 975	3 695	0.64	30	14	14	103	5
Double-tension structure................	23 500	19 850	0.86	51	16	28	135	2
SINGLE AND DOUBLE REVERSIBLES.								
Loading terminal.....	3 670	10 350	6.15	17	11	14.7	15	4
Tail tower.............	8 790	7 450	1.01	18	16	34	15	5

Cost of Erection.—Most tramway manufacturers supply the metal parts of the tramway in accordance with the usual terms of a material contract. The customer undertakes the erection and operation of the line, and he, therefore, finds it convenient to estimate the cost of erecting and installing the tramway. Tables 14 and 15 will be of service in this regard. It will be noted that the timber required is about 20% of the cost of the installation. Tables 9 and 12 indicate the timber contents of towers and stations, from which data the cost of the installation can be approximated.

TABLE 13.—AVERAGE WEIGHTS OF STEEL TRAMWAY STRUCTURES.

Name.	Volume, in cubic feet.	Length, in feet.	Width, in feet.	Height, in feet.	Weight, in pounds.	Weight, in cubic feet per pound.
Loading terminal............	18 260	67	22	15	37 000	2
2 bent structures............	10 860	16	16	40	17 000	1.6
Double anchorage and curve rail	25 900	68	16	26	67 000	2.5
Anchor and tension..........	19 650	45	16	25	63 000	3.2
Curve rail.....................	20 000	40	16	34	57 000	2.8
3 bent support	13 300	32	16	26	25 000	1.9
Double anchorage............	12 200	35	16	22	37 000	3.0
Discharge terminal	50 500	62	24	35	111 000	2.0
Control station...............	51 000	113	25	18	162 000	3.2

TABLE 14.—COST DATA, ERECTION OF TRAMWAYS.*

(Based on 30-ton line; 10 400 ft. long, with $\frac{7}{8}$-in. and $1\frac{1}{4}$-in. cables, and timber supports.)

Item.	Percentage.
Right of way	3.28
Trails, roads, and bridges	0.88
Excavations	3.31
Making forms	0.67
Setting forms, including engineering charges	5.44
Concreting	12.30
Back-filling	0.67
Distributing materials	11.10
Housing and roofing	8.10
Placing and assembling rolling stock	0.60
Stringing traction rope	1.88
" track cable	2.32
Placing cables in saddles	0.30
" " under tension	1.02
Framing towers	5.36
Erecting towers	5.43
Placing iron on towers	2.23
Framing loading terminal	1.68
Erecting loading terminal	1.90
Iron on loading terminal	1.95
Framing anchorage tower	1.61
Erecting anchorage tower	1.92
Iron on anchorage tower	0.71
Framing discharge terminal	1.96
Erecting discharge terminal	3.05
Iron on discharge terminal	1.15
Timber	19.70

* Installation charge equals 90% of cost of metal parts.

TABLE 15.—LABOR AND COST OF OPERATING BLEICHERT TRAMWAYS PER TON.

Capacity, in tons per hour.	Load terminal.	Discharge terminal.	Line rider and other help.	Total number of men.	Hourly wages @ $2.50 per day and 1 man @ $5.00, for 10 hours.	Cost of labor per ton.	Cost of supplies and renewals.	Cost of repairs.	Miscellaneous costs.	Total cost per ton.
10	1	1	..	2	$1.30	$0.13	$0.0235	$0.0447	$0.0200	$0.02182
25	1	1	1	3	1.80	0.072	0.0140	0.0260	0.0100	0.01220
40	2	1	1	4	2.30	0.0575	0.0120	0.0200	0.0080	0.0975
60	2	2	1	5	2.80	0.0467	0.0080	0.0160	0.0060	0.0767
75	3	2	1	6	3.30	0.0440	0.0075	0.0140	0.0058	0.0713
100	3	3	1	7	3.80	0.0380	0.0072	0.0134	0.0052	0.0638
150	4	3	1	8	4.30	0.0287	0.0054	0.0098	0.0040	0.0479
200	5	3	1	9	4.80	0.0240	0.0042	0.0078	0.0032	0.0392
250	5	4	1	10	5.30	0.0212	0.0040	0.0072	0.0028	0.0352

FORMULAS USED IN DESIGNING CABLE STRUCTURES

Notation.—The following list of symbols has been used by the writer:

a = carrier spacing.

A = area of cross-section of cable.

$$b = \frac{n\,(n-1)}{2}$$

B = distance between points of tangency of carriage wheels on cable.

$c = \dfrac{p\,(p-1)}{2}$

d = ratio of distance from support to first carrier to span $= \dfrac{m}{s} = d$.

e = base of natural system of logarithms.
E = modulus of elasticity.
f = coefficient of friction.
F = amount of friction in terminals.
g = weight of empty carrier.
G = weight of empty carrier plus load plus weight of traction rope of length, a.
h = difference in height of span supports.

J = change in length of cable for a unit load $= \dfrac{L}{A\,E}$.

l = length of cable in empty span.
L = length of cable in loaded span.
m = distance from support to first carrier.
n = total number of loads on span.
P = net load of the carrier.
p = number of loads to left of Point $x\ y$.
Q = capacity of tramway, in tons per hour.

q = ratio of carrier spacing to span $= \dfrac{a}{s} = q$.

R = radius of curvature.

r = ratio of distance to Point $x\ y$ to span $= \dfrac{x}{s} = r$.

s = horizontal length of span.
t = horizontal component of cable tension (empty spans).
T = horizontal component of cable tension (loaded spans).
tr = Traction rope tension, empty bucket side.
Tr = Traction rope tension, loaded bucket side.
v = velocity of traction rope, in units per minute.
w = weight of cable per unit of length.
W = weight of weight box.
x = horizontal distance from Y-axis to point under investigation.
y = distance from X-axis to point under investigation.
α = angle of slope from horizontal of chord joining supports of span.
β = angle of tangent from horizontal to curve at any point.

λ = elongation for cable for length and load $= \lambda = \dfrac{T\,L}{A\,E}$.

δ = time-spacing of carriers.
Δ = angle formed by intersecting tangents.
θ = angle of slope of tramway sections.
ϕ = angular change from end to end of catenary corresponds to Δ of parabolic formulas.

Using this notation, a number of formulas will be given, which will be found most useful in designing cable structures.

Empty Horizontal Span.—For a catenary with the origin at the left support (Fig. 9):

$$y = \frac{t}{w}\left[\sinh\frac{w\,x}{t}\sinh\frac{w\,s}{2\,t} - \cosh\frac{w\,s}{2\,t}\left(\cosh\frac{w\,x}{w\,t} - 1\right)\right] \ldots\ldots (1)$$

Deflection at center of span:

$$y = \frac{t}{w}\left(\cosh \frac{w\,s}{2\,t} - 1\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Example.—Let $s = 1\,000$ ft.; $t = 10\,000$ lb.; $w = 3.6$ ft-lb.; and $x = 500$ ft.; $y = ?$

$$\frac{t}{w} = 2\,777.7778; \ \cosh \frac{w\,s}{2\,t} = 1.0162$$

$$y = 2\,777.7778 \times 1.0162 = 2\,822.7778 - 2\,777.7778 = 45.0 \text{ ft.}$$

Inclined Span.—Use plus (+) for down, and minus (—) for up, slopes:

$$y = \left[\frac{t}{w} \sinh \frac{w\,x}{t} \sinh \frac{w\,s}{2\,t} - \cosh \frac{w\,s}{2\,t}\left(\cosh \frac{w\,x}{t} - 1\right)\right] \sec\,\alpha \pm x \tan\,\alpha. (3)$$

Example.—As in Equation (2), with $\tan\alpha = 0.1$; $\sec\alpha = 1.005$:

$$\begin{aligned} y = 45 \times 1.005 &= 45.225 \\ 500 \times 0.1 &= \underline{50.000} \\ &\ 95.000 \text{ ft.} \end{aligned}$$

Tangent.—

$$\frac{d\,y}{d\,x} = \left(\cosh \frac{w\,x}{t} \sinh \frac{w\,s}{2\,t} - \cosh \frac{w\,s}{2\,t} \sinh \frac{w\,x}{t}\right) \sec\,\alpha \pm \tan\,\alpha \ldots. (4)$$

Example.—$\tan\beta = ?$ in Equation (3).

At left support:

$$\tan\beta = \left(\sinh \frac{w\,s}{2\,t}\right) \sec\,\alpha \pm \tan\,\alpha \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

$$\sinh \frac{3.6 \times 1\,000}{20\,000} \sinh 0.18 = 0.1810$$

$$0.181 \sec\alpha = 0.181 \times 1.005 = 0.1819$$

$$0.1819 + \tan\alpha = 0.2819 = 15^\circ\,44' \text{ below horizontal.}$$

At right support:

$$0.1819 - 0.1 = 0.0819 = 4^\circ\,41' \text{ below horizontal.}$$

Length of Span.—

$$l = \frac{w}{t} \tan\,\phi \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which, $\tan\phi$ = angular change, end to end of span.

Example.—In Equation (6), $\tan\phi = 0.2819 + 0.0819$, or 2×0.1819:

$$l = 2\,777.78 \times 0.3638 = 1\,010.556 \text{ ft.}$$

If

$$l = \frac{w}{t} \tan\,\phi = \frac{2\,t}{w}\left(\sinh \frac{w\,s}{2\,t}\right)$$

$\sec\alpha$ is written with the expansion of $\sinh \frac{w\,s}{2\,t}$, then,

$$l = s + \frac{w^2\,s^3}{24\,t^2} \sec\,\alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

$$l = 1\,000.00 + (5.40 \times 1.005) = 1\,010.427 \text{ ft.}$$

This compares with the parabolic formula,

$$l = s + \frac{w^2\ s^3}{24\ t^2} + \frac{h^2}{2\ s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

$$\begin{array}{lcr} s \quad + & = & 1\,000.00 \\ \dfrac{w^2\ s^3}{24\ t^2} + & = & 5.40 \\ \dfrac{h^2}{2\ s} & = & 5.00 \\ \hline l & = & 1\,010.40 \text{ ft.} \end{array}$$

Simple Spans (Parabola).—Caution: If $\alpha >$ than 6°, multiply the right-hand side of the equation by sec α.

Deflection at any point of a simple inclined span (see Fig. 9):

$$y = \frac{w\ x}{2\ t}\ (s - x) \pm x \tan \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

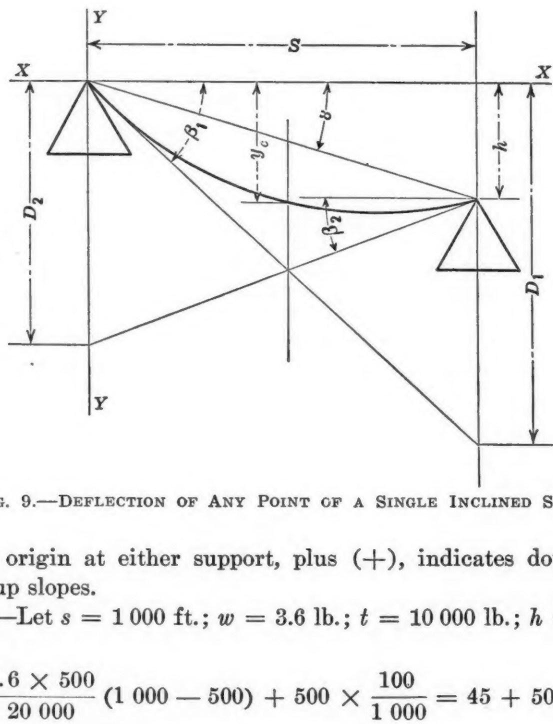

FIG. 9.—DEFLECTION OF ANY POINT OF A SINGLE INCLINED SPAN.

With the origin at either support, plus (+), indicates down slopes and minus (—), up slopes.

Example.—Let s = 1 000 ft.; w = 3.6 lb.; t = 10 000 lb.; h = 100 ft.; and x = 500 ft.

$$y = \frac{3.6 \times 500}{20\,000}\ (1\,000 - 500) + 500 \times \frac{100}{1\,000} = 45 + 50 = 95 \text{ ft.}$$

below the left-hand support.

Level Spans.—If the span is level, h = 0, and,

$$y = \frac{w\ x}{2\ t}\ (s - x) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

or 45 ft. in the previous example.

Note that with a center deflection of $\frac{1}{22.2}$ of the span, the catenary (Equation (2)) and parabola equations still give equal values.

Center of Span.—If $x = \frac{s}{2}$, then,

$$y = \frac{w\,s^2}{8\,t} \pm \frac{h}{2}$$

Note that $\frac{w\,s^2}{8\,t}$ is the vertical distance below the chord, and $\frac{h}{2}$ is the distance between the chord and the x-axis through the support.

If w equals the weight of a foot of cable, 1 sq. in. in cross-section, t equals pounds per square inch, and s equals hundreds of feet, then these simple equations for center deflection of empty spans result. (See Table 16.)

TABLE 16.—CENTER DEFLECTION OF EMPTY SPANS.

t, in pounds per square inch.	Deflection at center of span.	Example, 1 000-ft. span with 100-ft. difference in terminal elevation.
10 000	$y = 0.45s^2 + \frac{h}{2}$	$y\,c = 0.45 \times 10^2 + 50 = 95$ ft.
20 000	$y = 0.225s^2 + \frac{h}{2}$	$y\,c = 0.225 \times 10^2 + 50 = 72.5$ ft.
30 000	$y = 0.15s^2 + \frac{h}{2}$	$y\,c = 0.15 \times 10^2 + 50 = 65.0$ ft.
40 000	$y = 0.1125s^2 + \frac{h}{2}$	$y\,c = 0.1125 \times 10^2 + 50 = 61.25$ ft.
50 000	$y = 0.09s^2 + \frac{h}{2}$	$y\,c = 0.09 \times 10^2 + 50 = 59.0$ ft.

Deflection Formulas for Plotting Curve of Empty Cable.—If $y = \frac{w\,x}{2\,t}(s - x) + x \tan\alpha$ be stated as,

$$y - \frac{w\,s^2}{2\,t}\,r\,(l - r) + r\,h \ldots\ldots\ldots\ldots\ldots\ldots\ldots(11)$$

formulas of the type of Equation (11) may be written for different points along the span, as in Table 17.

Any other tension in pounds per square inch can be used by (inverse) proportion.

Example.—If $t = 10\,000$ lb. per sq. in.; $s = 1\,000$; $x = 300$; $y = ?$ In Table 17, at $x = 0.30$,

$$y = 9.45 + r\,h$$

The inverse ratio of tensions $= \frac{40\,000}{10\,000} = 4$; therefore, at 10 000,

$$y = 4 \times 9.45 = 37.80 + r\,h$$

Tangent of Simple Spans.—(See Fig. 9):

$$\tan\beta = \frac{d\,y}{d\,x} = \frac{w}{t}\left(\frac{s}{2} - x\right) \pm \tan\alpha$$

TABLE 17.—CURVE OF EMPTY CABLE.

(Deflections with $t = 40\,000$ lb. per sq. in.; $y = 0.45s^2r\ (l - r)$; s given in hundreds of feet.)

$\frac{x}{s} = r.$	y at $t = 40\,000$ lb. per sq. in.	$\frac{x}{s} = r.$	y at $t = 40\,000$ lb. per sq. in.
0.02, or 0.98	$0.00882\ s^2 + r\,h$	0.26, or 0.74	$0.08658\ s^2 + r\,h$
0.04, or 0.96	$0.01728\ s^2 + r\,h$	0.28, or 0.72	$0.09072\ s^2 + r\,h$
0.05, or 0.95	$0.021375\ s^2 + r\,h$	0.30, or 0.70	$0.09450\ s^2 + r\,h$
0.06, or 0.94	$0.02538\ s^2 + r\,h$	0.32, or 0.68	$0.09792\ s^2 + r\,h$
0.08, or 0.92	$0.03312\ s^2 + r\,h$	0.34, or 0.66	$0.10098\ s^2 + r\,h$
0.10, or 0.90	$0.0405\ s^2 + r\,h$	0.35, or 0.65	$0.102375\ s^2 + r\,h$
0.12, or 0.88	$0.04752\ s^2 + r\,h$	0.36, or 0.64	$0.10368\ s^2 + r\,h$
0.14, or 0.86	$0.05418\ s^2 + r\,h$	0.38, or 0.62	$0.10602\ s^2 + r\,h$
0.15, or 0.85	$0.057375\ s^2 + r\,h$	0.40, or 0.60	$0.10800\ s^2 + r\,h$
0.16, or 0.84	$0.06048\ s^2 + r\,h$	0.42, or 0.58	$0.10962\ s^2 + r\,h$
0.18, or 0.82	$0.06642\ s^2 + r\,h$	0.44, or 0.56	$0.11088\ s^2 + r\,h$
0.20, or 0.80	$0.07200\ s^2 + r\,h$	0.45, or 0.55	$0.111375\ s^2 + r\,h$
0.22, or 0.78	$0.07722\ s^2 + r\,h$	0.46, or 0.54	$0.11178\ s^2 + r\,h$
0.24, or 0.76	$0.08208\ s^2 + r\,h$	0.48, or 0.52	$0.11232\ s^2 + r\,h$
0.25, or 0.75	$0.084375\ s^2 + r\,h$	0.50, or 0.50	$0.11250\ s^2 + r\,h$

Example.—If $s = 1\,000$ ft.; $t = 40\,000$ lb.; $w = 3.6$ lb.; $\tan\alpha = 0.1$; $x = 300$ ft.; $\tan\beta = ?$

$\tan\beta = 0.00009 \times 200 + 0.1 = 0.1180$; $\beta = 6°\ 44'$ below horizontal axis

Tangent at Supports.—Left, $x = 0$; right, $x = s$:

Left:

$$\tan\beta_1 = \frac{w\ s}{2\ t} \pm \tan\alpha \text{ (+ when } \alpha \text{ is below horizontal axis)}$$

Example.—If $s = 1\,000$; $t = 40\,000$ lb.; $w = 3.6$ lb.; $\tan\alpha = 0.1$:

$\tan\beta_1 = 0.00009 \times 500 + 0.1 = 0.1450$; $\beta_1 = 8°\ 15'$ below horizontal axis

Right:

$$\tan\beta_2 = \frac{w\ s}{2\ t} \pm \tan\alpha \text{ (+ when } \alpha \text{ is above horizontal axis)}$$

Example.—As before:

$\tan\beta_2 = -\ 0.00009 \times 500 + 0.1 = 0.055 = 3°\ 09'$ above horizontal axis

The tangent formula is of a type that is easily adapted to any span; thus, if $t = 40\,000$ lb. per sq. in., $w = 3.6$ lb., and s is in feet, there results:

$$\frac{w\ s}{2\ t} \pm \tan\alpha = 0.000045\ s \pm \tan\alpha$$

TABLE 18.—VALUES OF 0.000045 s FOR TENS OF FEET.

Span, in feet.	10	20	30	40	50	60	70	80	90
$\frac{w\ s}{2\ t}$	0.00045	0.00090	0.00135	0.0018	0.00225	0.0027	0.00315	0.0036	0.00405

A table, such as Table 18, may be used for finding the tangent at the support of any simple span. Thus, let s = 545 ft.; t = 40 000 lb. per sq. in.; and $\tan \alpha = 0.1$:

$\tan \beta_1 = 0.0225$

0.0018 = 0.0245 + 0.1 = 0.1245; $\beta_1 = 7° \ 06'$ below the horizontal axis

0.0002

If t was 20 000 lb. per sq. in., then,

$$\beta_1 = \frac{40\ 000}{20\ 000} \times 0.0245 + 0.1 = 0.149\,;\ \beta_1 = 8° \ 29'$$

Maximum Span for Limiting Value of $\tan \beta_1$.—As it is often found advisable to limit the angle over the supports of a span, the length of such a span may be found by a reverse process. Thus, let $\tan \beta = 0.25$ and t = 40 000 lb. per sq. in. Taking from Table 18, the successive spans corresponding to the nearest smaller values of $\frac{w\ s}{2\ t}$, the total horizontal span may be found as follows:

$$\text{For } \frac{w\ s}{2\ t} = 0.225, \text{ span } = 5\ 000 \text{ ft.}$$

$$\text{For } \frac{w\ s}{2\ t} = 0.0225, \text{ span } = 500 \text{ ft.}$$

$$\text{For } \frac{w\ s}{2\ t} = 0.00225, \text{ span } = 50 \text{ ft.}$$

$$\text{For } \frac{w\ s}{2\ t} = 0.000225, \text{ span } = 5 \text{ ft.}$$

$$\text{For } \frac{w\ s}{2\ t} = 0.249975, \text{ span } = 5\ 555 \text{ ft.}$$

By the formula, for the horizontal axis,

$$s = \tan \beta_1 \times \frac{2\ t}{w} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(12)}$$

$$= 0.25 \times 22\ 222.22 = 5\ 555.5 \text{ ft.}$$

By the formula, for the inclined axis,

$$s = \frac{t}{w} \tan \beta_1 \pm \frac{1}{w} \sqrt{t^2 \tan^2 \beta_1 - 2\ w\ h\ t} \ldots\ldots\ldots\ldots \text{(13)}$$

Example.—In these data, let h = 100 ft.; then,

$$s = 0.25 \times 11\ 111.11 + 0.2778 \sqrt{100\ 000\ 000 - 28\ 800\ 000}$$

$$= 2\ 778 + 2\ 344 = 5\ 122 \text{ ft.}$$

This may be proven by correcting the value of $\tan \beta_1$ by $\tan \alpha_1$ and solving as a horizontal span. Thus,

$$\tan \alpha = \frac{100}{5\ 122} = 0.0195$$

$$\tan \beta_1 = 0.25 - 0.0195 = 0.2305$$

$$s = 0.2305 \times 22\ 222.22 = 5\ 122 \text{ as before}$$

Lowest Point of a Simple Span.—(See Fig. 9):

If $\tan \beta = 0$,

$$x = \frac{s}{2} + \frac{t}{w} \tan \alpha \quad \text{(14)}$$

If, $\tan \beta = \tan \alpha$,

$$x = \frac{s}{2} \quad \text{(15)}$$

Example.—If $s = 1\,000$; $w = 3.6$; $t = 40\,000$ lb.; and $\tan \alpha = 0.0100$; how far from the left support is the lowest point of the cable?

$$x = 500 + 111.11 = 611.11 \text{ ft.}$$

If the ground surface is level, the minimum clearance will be found at this point. If x is greater than the span, the right-hand support is the lowest point.

Rate of Curvature.—

$$\frac{d^2 y}{d x^2} = -\frac{w}{t} \quad \text{(16)}$$

If, $w = 3.6$ lb. and $t = 40\,000$ lb.,

$$\frac{w}{t} = 0.00009 = \text{tan of angle per foot of span}$$

For 100 ft., $0.009 = 0° \; 31'$ as the total angular change for a 100-ft. span. If horizontal, this angle is equally divided between β_1 and β_2; that is, the slope of the tangent at the left support is altered 15.5′ for every 100 ft. of span for a cable, $w = 3.6$ lb. and $t = 40\,000$ lb. This permits computing the value of β_1 by direct multiplication. Thus, a 1 000-ft. span $= 10 \times 15.5' = 2° \; 35' = \beta_1$, if $t = 40\,000$ lb., or $10 \times 20.7' = 3° \; 27' = \beta_1$, if $t = 30\,000$ lb.

The Tangent Intercept.—Simple Spans.—(See Fig. 9):

$$D_1 = s \times \tan \beta_1 = \frac{w \, s^2}{2 \, t} + h \quad \text{(17)}$$

But, $\frac{w \, s^2}{2 \, t} = 4 \times$ the deflection of the center of a horizontal span.

Example.—If $s = 1\,000$ ft.; $t = 40\,000$ lb. per sq. ft.; $h = 100$ ft.; $D_1 = $? From Table 16 (deflections at 40 000 lb. per sq. ft.),

$$yc = (11.25 \times 4) + 100 = D_1$$

$$D_2 = h + s \tan \beta_2 = h + s \left(\frac{w \, s}{2 \, t} - \tan \alpha \right) = \frac{w \, s^2}{2 \, t} \quad \text{(18)}$$

Point of Intersection.—The equations of Line (1) and Line (2) (Fig. 9) may be written:

$$y = x \left(\frac{w \, s}{2 \, t} + \tan \alpha \right) \quad \text{(19)}$$

$$y = + \frac{w \, s^2}{2 \, t} - x \left(\frac{w \, s}{2 \, t} - \tan \alpha \right) \quad \text{(20)}$$

Subtracting Equation (20) from Equation (19),

$$0 = \frac{w\,s\,x}{2\,t} + x \tan \alpha - \frac{w\,s^2}{2\,t} + \frac{w\,s\,x}{2\,t} - x \tan \alpha \ldots\ldots\ldots\ldots (21)$$

or,

$$\frac{2\,w\,s\,x}{2\,t} = \frac{w\,s^2}{2\,t};\ x = \frac{s}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Therefore, the tangent rays always intersect $x = \frac{s}{2}$.

The distance below the left support is,

$$\frac{s}{2} \tan \beta_1 = \frac{s}{2}\left(\frac{w\,s}{2\,t} + \tan \alpha\right) = \frac{w\,s^2}{4\,t} + \frac{h}{2} \ldots\ldots\ldots\ldots (23)$$

Note that $\frac{w\,s^2}{4\,t}$ equals twice the center deflection of a horizontal span. Since the point of intersection and the tangent intercepts are so readily determined, the tangents to the curve at its supports are easily drawn or computed.

Radius of Curvature.—The well-known formula for radius of curvature is,

$$R = \frac{\left[1 + \left(\frac{d\,y}{d\,x}\right)^2\right]^{\frac{3}{2}}}{\frac{d^2\,y}{d\,x^2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

For a horizontal span, with $x = \frac{s}{2}$ and $\frac{d\,y}{d\,x} = 0$,

$$R = \frac{1}{-\frac{w}{t}} = -\frac{t}{w} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (25)$$

This permits the use of circular arcs to represent cable curves. Thus, let the profile be plotted on a scale of 1 in. = 100 ft., what radius curve should be used for a cable, $w = 3.6$ lb. and $t = 40\,000$ lb.?

$$R = -\frac{t}{w} = 11\,111.11 \text{ ft.}$$

But 1 in. = 100 ft. Therefore, the proper curve is $R = 111.1$ in., if $t = 30\,000$ lb.; $\frac{t}{w} = 8\,333.3$ ft.; and $R = 83.33$ in.

Length of Simple Spans.—(See Fig. 9):

$$l = s + \frac{w^2\,s^3}{24\,t^2} + \frac{h^2}{2\,s} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

Example.—If $s = 1\,000$ ft.; $w = 3.6$ lb.; $h = 100$ ft.; and $t = 40\,000$ lb.; $l = ?$

$$s = 1\,000 \text{ ft.}$$

$$\frac{w^2\,s^3}{24\,t^2} = 0.338$$

$$\frac{h^2}{2\,s} = 5.00$$

Then,

$$l = 1\,005.338$$

Carrier Spacing, in Feet.—

Given $P =$ the net load of a carrier.

$Q =$ capacity, in tons.

$V =$ velocity, in feet per minute.

$$a = \frac{0.03\ P\ V}{Q} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(27)}$$

Carrier Spacing, in Seconds.—

$$\phi = \frac{a}{V}\ 60 = \frac{1.8\ P}{Q} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(28)}$$

Example.—$P = 1\,000$ lb.; $Q = 20$ tons per hour; and $V = 500$ ft. per min.:

$$a = \frac{15 \times 1\,000}{20} = 7.50 \text{ ft.}$$

$$\phi = \frac{1.8 \times 1\,000}{20} = 90 \text{ sec.}$$

Cable Spans Supporting Carriers.—Span Carrying a Single Load.—Note that a cable supporting loads does not present a continuous curve, but rather $(n + 1)$ parabolas which intersect with an angle, $\Delta = \tan^{-1} \frac{G}{t}$, beneath each load. (See Fig. 10.) The curve of the path of any load is continuous and may be treated accordingly.

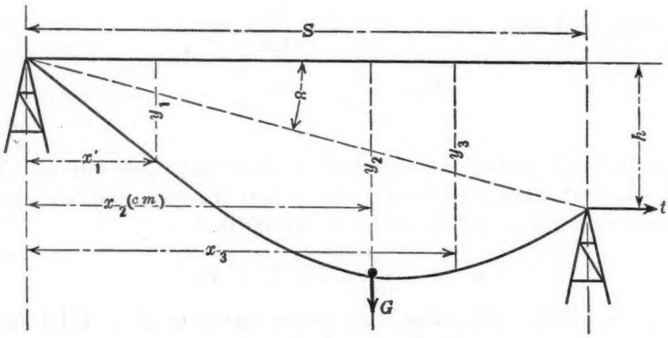

FIG. 10.—SPAN CARRYING A SINGLE LOAD.

Case I.—$m > x < s$ = equation of parabola adjoining left support:

$$y_1 = \frac{G\ (s - m)\ x_1}{s\ t} + \frac{w\ x_1}{2\ t}\ (s - x_1) + x_1 \tan \alpha \ldots\ldots\ldots\ldots \text{(29)}$$

Case II.—$m = x < s$ = equation for path of load:

$$y_2 = \frac{G\ x_2\ (s - x_2)}{s\ t} + \frac{w\ x_2\ (s - x_2)}{2\ t} + x_2 \tan \alpha \ldots\ldots\ldots\ldots \text{(30)}$$

Case III.—$m < x < s$ = equation of parabola adjoining right support:

$$y_3 = \frac{G\ m\ (s - x_3)}{s\ t} + \frac{w\ x_3\ (s - x_3)}{2\ t} + x_3 \tan \alpha \ldots\ldots\ldots\ldots \text{(31)}$$

When $x = m = \frac{s}{2}$, Equations (29), (30), and (31) become,

$$y = \frac{G\ s}{4\ t} + \frac{w\ s^2}{8\ t} + \frac{h}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(32)}$$

Example.—Let $s = 1\,000$ ft.; $m = 500$ ft.; $x_1 = 200$ ft.; $x_2 = 500$ ft.; $x_3 =$ 800 ft.; $t = 75\,000$ lb.; $w = 10$ lb. per ft.; $h = 100$ ft.; and $G = 10\,000$ lb.; y_1, y_2, and $y_3 =$?

$$y_1 = \frac{10\,000 \times 500 \times 200}{1\,000 \times 75\,000} + \frac{10 \times 200 \times 800}{150\,000} + 200 \times \frac{100}{1\,000} =$$

$$13.33 + 10.67 + 20 = 44 \text{ ft. below left support.}$$

$$y_2 = \frac{10\,000 \times 500 \times 500}{1\,000 \times 75\,000} + \frac{10 \times 500 \times 500}{150\,000} + \frac{500 \times 100}{1\,000} =$$

$$33.333 + 16.667 + 50 = 100 \text{ ft. below left support.}$$

$$y_3 = \frac{10\,000 \times 500 \times 200}{1\,000 \times 75\,000} + \frac{10 \times 800 \times 200}{150\,000} + \frac{800 \times 100}{1\,000} =$$

$$13.33 + 10.67 + 80 = 104 \text{ ft. below left support.}$$

Tangents to Span Supporting One Load.—(See Fig. 10):

Case I.—

$$\tan \beta = \frac{d\,y}{d\,x} = \frac{w\,s}{2\,t} - \frac{w\,x}{t} + \frac{G}{s\,t}(s - m) + \tan \alpha \quad \text{.......... (33)}$$

If $x_1 = 0$, $m = 0$,

$$\tan \beta_1 = \frac{w\,s}{2\,t} + \frac{G}{t} + \tan \alpha = \text{tangent with load at left support} \quad \text{.....(34)}$$

Case II.—

$$\tan \beta = \frac{w\,s}{2\,t} - \frac{w\,x_2}{t} + \frac{G}{t} - \frac{2\,G\,x_2}{s\,t} \pm \tan \alpha \quad \text{.............(35)}$$

If $x_2 = 0$, $m = 0$,

$$\tan \beta_1 = \frac{w\,s}{2\,t} + \frac{G}{t} \pm \tan \alpha \quad \text{.................... (36)}$$

If $x_2 = s$, $m = s$,

$$\tan \beta_2 = \frac{w\,s}{2\,t} + \frac{G}{t} \mp \tan \alpha \quad \text{....................(37)}$$

Case III.—

$$\tan \beta = \frac{w\,s}{2\,t} - \frac{w\,x_3}{t} - \frac{G\,m}{s\,t} \mp \tan \alpha \quad \text{................(38)}$$

If $x_3 = s$, $m = s$,

$$\tan \beta_2 = -\frac{w\,s}{2\,t} - \frac{G}{t} \mp \tan \alpha = \frac{w\,s}{2\,t} + \frac{G}{t} \mp \tan \alpha \quad \text{......... (39)}$$

Example.—If $s = 1\,000$ ft.; $m = 500$ ft.; $x_1 = 200$ ft.; x_2 500 ft.; $x_3 =$ 800 ft.; $t = 75\,000$ lb.; $w = 10$ lb. per ft.; $h = 100$ ft.; and $G = 10\,000$ lb.; $\tan \beta =$?

$$\tan \beta_1 = \frac{10 \times 1\,000}{2 \times 75\,000} - \frac{10 \times 200}{75\,000} + \frac{10\,000}{1\,000 \times 75\,000}(500) + 0.1$$

$$= 0.0667 - 0.0267 + 0.0667 + 0.1 = 0.2067$$

Therefore, the tangent to the cable 200 ft. from the left support makes an angle of 11° 41′ with the horizontal.

$$\tan \beta_1 \text{ at support} = 0.0667 + 0.1333 + 0.1 = 0.3000 = 16° \ 42'$$

$$\tan \beta_2 = - 0.0667 - 0.0667 + 0.1333 + 0.1 = 0.1000 = 5° \ 43'$$

$$\tan \beta_3 = 0.0667 - 0.1067 - 0.1066 + 0.1 = - 0.0466 = - 2° \ 40'$$

It is understood that when using Equations (33) to (39) the minus signs indicate below horizontal, and the plus signs, above horizontal.

Spans Supporting More Than One Load.—(See Fig. 11 and Table 19):

$$y = \frac{G}{t} \left\{ x (n - p) - m \left[\frac{x n}{s} - p\right] - a \left[\frac{b x}{s} - c\right] \right\}$$

$$+ \frac{w x}{2 t} (s - x) \pm x \tan \alpha \ldots\ldots\ldots\ldots\ldots\ldots (40)$$

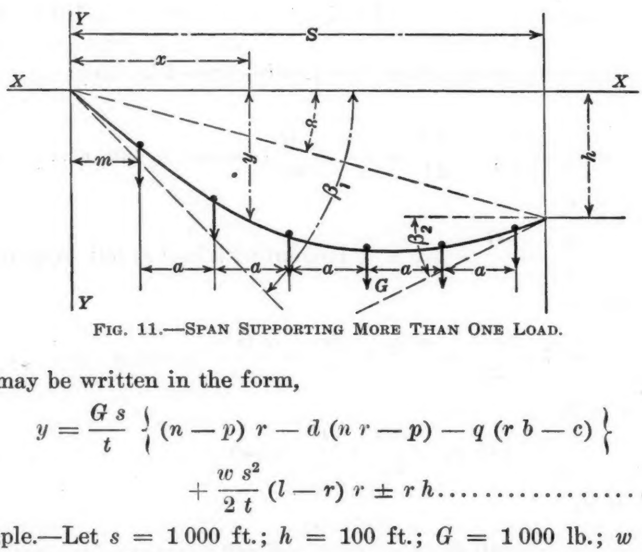

FIG. 11.—SPAN SUPPORTING MORE THAN ONE LOAD.

This may be written in the form,

$$y = \frac{G s}{t} \left\{ (n - p) r - d (n r - p) - q (r b - c) \right\}$$

$$+ \frac{w s^2}{2 t} (l - r) r \pm r h \ldots\ldots\ldots\ldots\ldots\ldots (41)$$

Example.—Let $s = 1\,000$ ft.; $h = 100$ ft.; $G = 1\,000$ lb.; $w = 3.6$ lb.; $t = 40\,000$ lb.; $n = 4$; $a = 300$ ft.; $m = 50$ ft.; and $x = 500$ ft.; $y = ?$

$$r = \frac{500}{1\,000} = 0.5; \ d = \frac{50}{1\,000} = 0.05; \ q = \frac{300}{1\,000} = 0.3$$

$$y = 25 \left\{ 2 \times 0.5 - 0.05 (4 \times 0.5 - 2) - 0.3 (0.5 \times 6 - 1) \right\}$$

$$+ \frac{3.6 \times 1\,000^2}{80\,000} 0.5 \times 0.5 + 0.5 \times 100$$

$$y = 10 + 11.25 + 50 = 71.25 \text{ ft. below left support}$$

Tangent to Span Supporting Multiple Loads.—(See Fig. 11):

$$\tan \beta = \frac{d y}{d x} = \frac{G}{t} \left\{ (n - p) - \frac{n m + b a}{s} \right\} + \frac{w}{t} \left(\frac{s}{2} - x\right) \pm \tan \alpha$$

which may be written,

$$\tan \beta = \frac{G}{t} [(n - p) - d n - q b] + \frac{w s}{t} \left(\frac{1}{2} - r\right) \pm \tan \alpha \ldots (42)$$

Example.—Data as in previous example.

At the left support:

$$\tan \beta_1 = 0.025\,[4 - 0.20 - 1.8] + 0.027 + 0.1$$
$$= 0.050 + 0.027 + 0.1 = 0.177 = 10^\circ\ 02'$$

At the center:

$$\tan \beta_3 = 0.025\,[2 - 0.20 - 1.8] + 0 + 0.1 = 0.1 = 5^\circ\ 43'$$

At the right support:

$$\tan \beta_2 = 0.025\,[0 - 0.2 - 1.8] - 0.027 + 0.1 = 0.023$$
$$= 1^\circ\ 19' \text{ below horizontal axis.}$$

TABLE 19.—Deflection at Center of Spans Supporting Multiple Loads, Symmetrically Placed.

No. of loads.	Deflection of loads.	+ of cable.	$+\frac{h}{2}$
1	$\frac{G\,s}{4\,t}$	$\frac{w\,s^2}{8\,t}$	$+\frac{h}{2}$
2	$\frac{G\,s}{4\,t}(2 - 2\,q)$	$\frac{w\,s^2}{8\,t}$	$+\frac{h}{2}$
3	$\frac{G\,s}{4\,t}(3 - 4\,q)$	$\frac{w\,s^2}{8\,t}$	$+\frac{h}{2}$
4	$\frac{G\,s}{4\,t}(4 - 8\,q)$	$\frac{w\,s^2}{8\,t}$	$+\frac{h}{2}$
5	$\frac{G\,s}{4\,t}(5 - 12\,q)$	$\frac{w\,s^2}{8\,t}$	$+\frac{h}{2}$
6	$\frac{G\,s}{4\,t}(6 - 18\,q)$	$\frac{w\,s^2}{8\,t}$	$+\frac{h}{2}$
7	$\frac{G\,s}{4\,t}(7 - 24\,q)$	$\frac{w\,s^2}{8\,t}$	$+\frac{h}{2}$
8	$\frac{G\,s}{4\,t}(8 - 32\,q)$	$\frac{w\,s^2}{8\,t}$	$+\frac{h}{2}$
9	$\frac{G\,s}{4\,t}(9 - 40\,q)$	$\frac{w\,s^2}{8\,t}$	$+\frac{h}{2}$
10	$\frac{G\,s}{4\,t}(10 - 50\,q)$	$\frac{w\,s^2}{8\,t}$	$+\frac{h}{2}$

With an even number of loads, the deflection under load to right of center is,

$$\frac{G\,s}{4\,t}\left[N - \frac{N^2}{2}\,q\right] + \frac{w\,s^2}{8\,t}\,(1 - q^2) \pm r\,h \ldots\ldots\ldots\ldots\ldots (43)$$

With an odd number of loads, the deflection under the center load is,

$$\frac{G\,s}{4\,t}\left[N - \frac{N^2 - 1}{2}\,q\right] + \frac{w\,s^2}{8\,t} \pm \frac{h}{2} \ldots\ldots\ldots\ldots\ldots (44)$$

Example.—Let $s = 1\,000$ ft.; $h = 100$ ft.; $G = 1\,000$ lb.; $w = 3.6$ lb.; $n = 4$; $t = 40\,000$ lb.; and $a = 300$ ft.; y at center = ?

$$q = \frac{300}{1\ 000} = 0.3\,;\ m = 50 \text{ ft.}$$

$$y = \frac{1\,000 \times 1\,000}{4 \times 40\,000}\,[4 - 8 \times 0.3] + \frac{3.6 \times 1\,000^2}{8 \times 40\,000} + 50$$
$$= 6.25 \times 1.6 + 11.25 + 50 = 71.25 \text{ ft.}$$

below left support.

Note that with n equal to an odd number, the middle load is at the center of the span.

Tangents Due to Loads Only on Saddle of Adjoining Multiple-Loaded Spans.—In Fig. 12 let the loads be arranged so that one is at the support, B. Also, $q = \frac{a}{s}$, q_1 = Span A B, and q_2 = Span B C (Table 20).

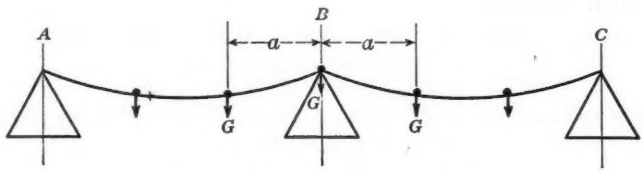

FIG. 12.—MULTIPLE-LOADED SPANS.

TABLE 20.

Span A B.	No. of loads.	Span B C.
$\frac{G}{t}(1 - q_1)$	1	$\frac{G}{t}(1)$
$\frac{2\,G}{t}(1 - \frac{3}{2}\,q_1)$	2	$\frac{2\,G}{t}(1 - 0.5\,q_2)$
$\frac{3\,G}{t}(1 - 2\,q_1)$	3	$\frac{3\,G}{t}(1 - q_2)$
$\frac{4\,G}{t}(1 - 2.5\,q_1)$	4	$\frac{4\,G}{t}(1 - 1.5\,q_2)$
$\frac{5\,G}{t}(1 - 3\,q_1)$	5	$\frac{5\,G}{t}(1 - 2\,q_2)$
$\frac{6\,G}{t}(1 - 3.5\,q_1)$	6	$\frac{6\,G}{t}(1 - 2.5\,q_2)$
$\frac{7\,G}{t}(1 - 4\,q_1)$	7	$\frac{7\,G}{t}(1 - 3\,q_2)$
$\frac{8\,G}{t}(1 - 4\ 5\,q_1)$	8	$\frac{8\,G}{t}(1 - 3.5\,q_2)$
$\frac{9\,G}{t}(1 - 5\,q_1)$	9	$\frac{9\,G}{t}(1 - 4\,q_2)$
$\frac{10\,G}{t}(1 - 5.5\,q_1)$	10	$\frac{10\,G}{t}(1 - 4.5\,q_2)$

Example.—If Span $A\,B$ = 900 ft. and Span $B\,C$ = 1 000 ft., a = 300 ft.; G = 1 000 lb.; and t = 40 000 lb., the maximum angle over Support B due to loads only, can be found, as follows:

Span $A\,B$ = 3 loads = 3 × 0.025 (1 — 0.667) = 0.025 = 1° 26′
Span $B\,C$ = 4 loads = 4 × 0.025 (1 — 0.45) = 0.055 = 3° 09′

Maximum angle over Support B = 4° 35′

Catenary.—The deflection of a loaded cable when treated as a catenary is:

$$y = \frac{G\,s}{t}\left[(n - p)\,r - d\,(n\,r - p) - q\,(b\,r - c)\right] + \frac{t}{w}\left[\sinh\frac{w\,x}{t}\sinh\frac{w\,s}{2\,t} - \cosh\frac{w\,s}{2\,t}\left(\cosh\frac{w\,x}{t} - 1\right)\right]\sec\alpha \pm r\,h \qquad (45)$$

At the center of span:

$$y = \frac{G\,s}{4\,t}\left[n - \frac{n^2 - 1}{2}\,q\right] + \frac{t}{w}\left[\cosh\frac{w\,s}{2\,t} - 1\right]\sec\alpha \pm \frac{h}{2} \ldots\ldots (46)$$

At the tangent:

$$\frac{d\,y}{d\,x} = \tan\beta = \frac{G}{t}\,[(n - p) - d\,n - q\,b] + \left(\cosh\frac{w\,x}{t}\sinh\frac{w\,s}{2\,t} - \cosh\frac{w\,s}{2\,t}\sinh\frac{w\,x}{t}\right)\sec\alpha \pm \tan\alpha \ldots\ldots\ldots\ldots\ldots\ldots (47)$$

The deflections of a cable span, when loaded symmetrically, can be computed rapidly, by the method of differences.* Use is made of the fact that all the second differences of the deflections are equal to $-\frac{a}{t}\,(G + w\,a)$. Further, if the number of loads on the span is odd, the first differences are numerically equal to one-half the second. If the number is even, then the first differences are equal to the second. The slope correction is constant, and is to be added or subtracted from the first differences as indicated by the direction of the slope of the chord. In using this method it is necessary only to compute one deflection (which may be for any load, but preferably a center one), because of the simplicity of the calculation, and find the others by addition, thus: Let $s = 1\,000$ ft.; $h = 100$ ft.; $G = 1\,000$ lb.; $w = 3.6$ lb.; $t = 40\,000$ lb.; $a = 300$ ft.; and $m = 50$ ft.; $y = ?$

Compute the deflection of load nearest the left support:

$$y = \frac{G\,s\,r}{t}\left[n\,(1 - d) - q\,b\right] + \frac{w\,s^2\,r}{2\,t}(1 - r) \pm r\,h \ldots\ldots\ldots (48)$$

$$\frac{1\,000 \times 1\,000 \times 0.05}{40\,000}\left[4\,(1 - 0.05) - \frac{0.3 \times 4 \times 3}{2}\right] +$$

$$\frac{3.6 \times 1\,000 \times 0.05 \times 0.95}{80\,000} + 0.05 \times 100 = 2.50 + 2.14 + 5 = 9.64 \text{ ft.}$$

The second difference is:

$$-\frac{300}{40\,000} - (1\,000 + 1\,080) = -15.6 \text{ ft.}$$

The first difference is:

$$15.6 + 0.3 \times 100 = 45.6 \text{ ft.}$$

Construct Table 21, writing the deflection found for Load 1; then add the first difference to it for the next load; then correct the first difference by subtracting the second difference and the new first difference is obtained; which, added to the deflection of the second load gives the third, etc.

TABLE 21.

Load No.	1		2		3		4
Deflection	9.64		55.24		85.24		99.64
First difference		45.6		30		14.40	
Second difference			−15.6		−15.6		

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1383.

The deflections of loaded spans may also be quickly ascertained by the use of Figs. 13, 14, and 15 for: (1) the slope factor; (2) the deflection of empty cable; and (3) the deflection due to load. Their use may be illustrated by the following example: Let, $s = 1\,000$ ft.; $h = 200$ ft.; $g = 2\,800$ lb.; $t = 60\,000$ lb.; $w = 5.5$ lb.; $X = 100$ ft.; and $M = 300$ ft.; find y.

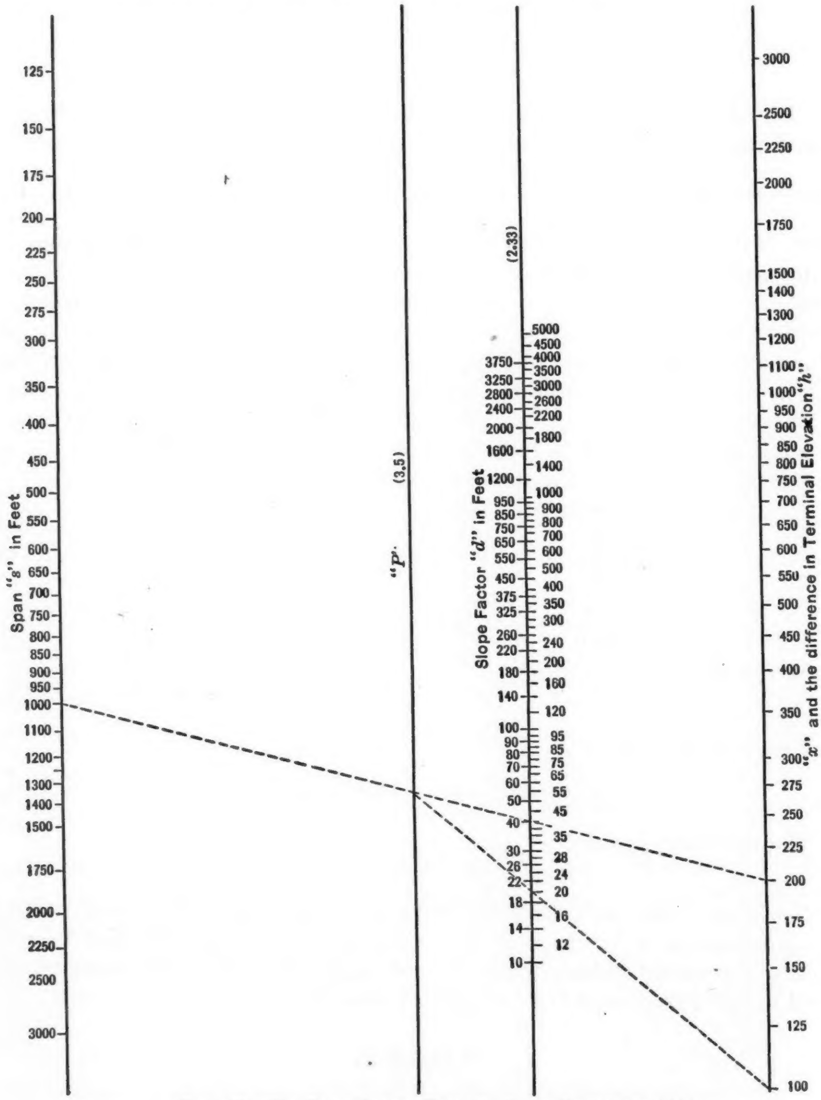

FIG. 13.—TO FIND SLOPE FACTOR, d, IN EQUATION (49).

(a) Refer to Fig. 13 and solve for the slope factor:

$$d = \frac{x}{s} h \quad \text{.......................... (49)}$$

First, place the straight-edge from s to h and mark p. Then place the straight-edge from p to x and read the intercept, d.

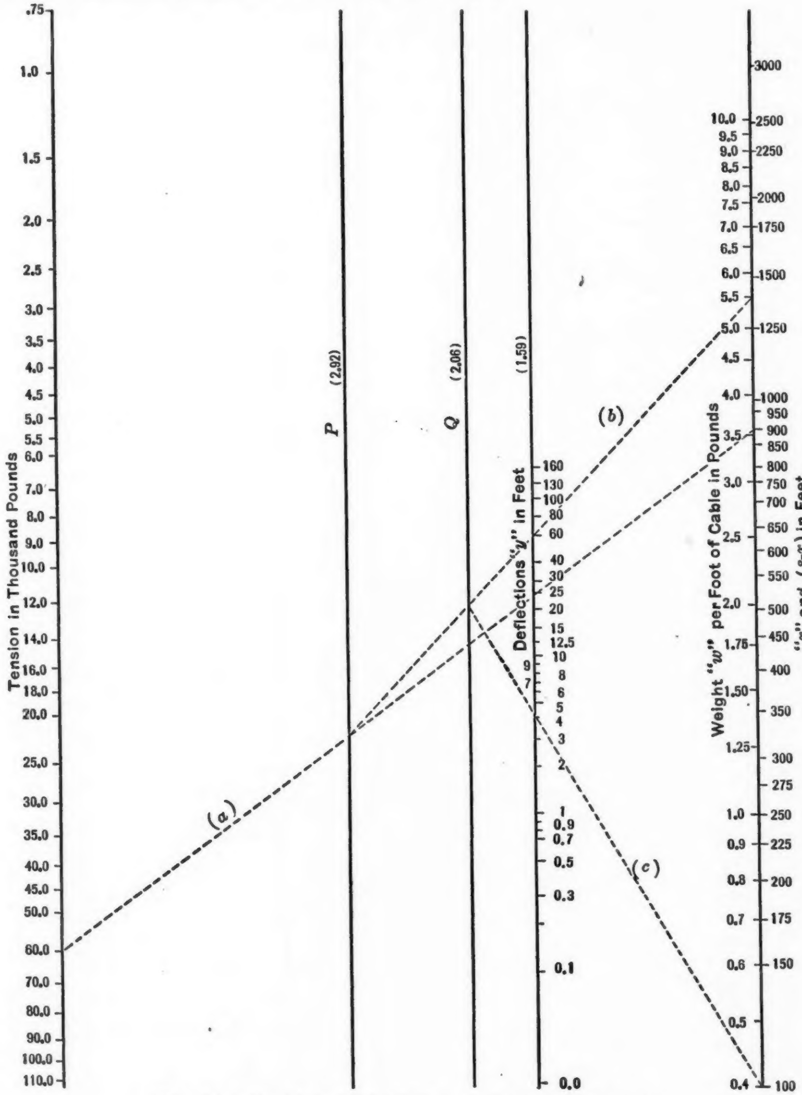

FIG. 14.—TO FIND DEFLECTION, y, IN EQUATION (50).

Thus,

$$d = \frac{100 \times 200}{1\ 000} = 20 \text{ ft.}$$

(b) Refer to Fig. 14 and solve for,

$$y = \frac{w\, x\, (s - x)}{2\, t} \quad \ldots\ldots\ldots\ldots\ldots\ldots (50)$$

First, place the straight-edge on the given tension and on x, or $(5\,x)$ and note the intersection on p. Next, place the straight-edge from this point to the weight, w, and note the intersection on q. Finally, place the straight-edge from this point (on q) to $(s - x)$, or x, and read the deflection on y.

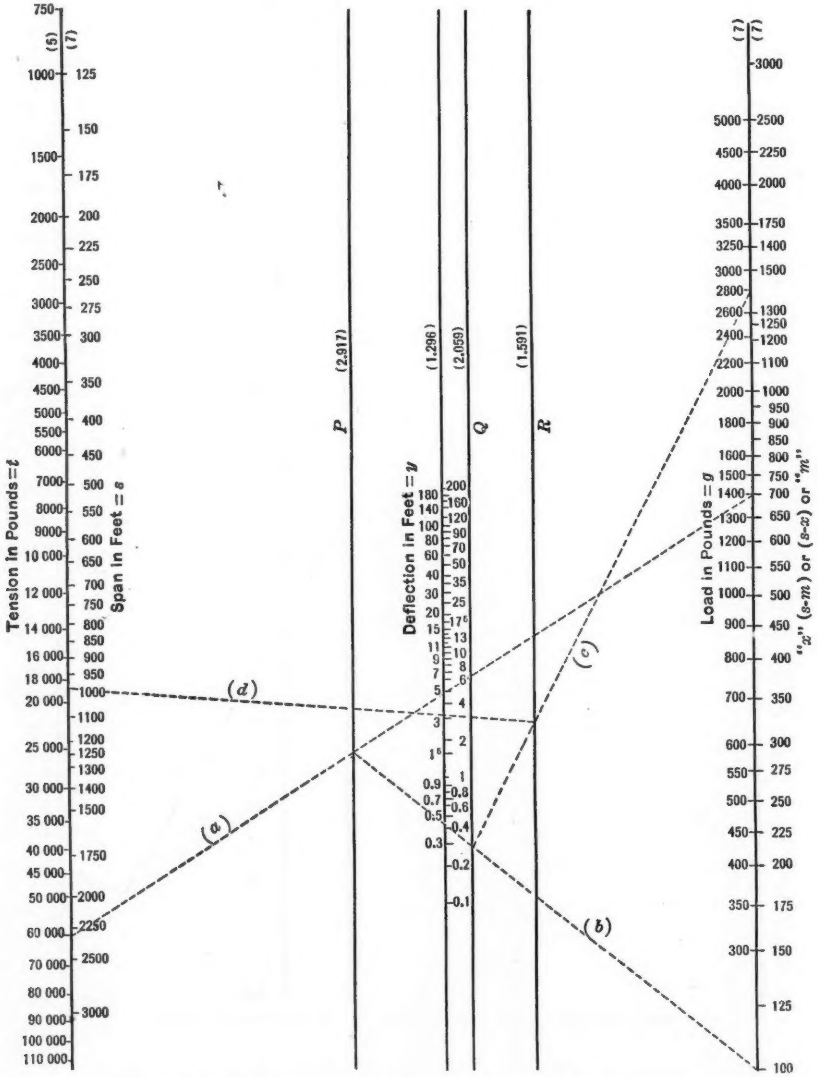

FIG. 15.—TO FIND THE DEFLECTION, y, FOR SPECIAL CASES.

Thus,

$$y = \frac{5.5 \times 100 \times 900}{2 \times 60\,000} = 4.1 \text{ ft.}$$

Equation (50) becomes:

$$y = \frac{g\,x\,(s - m)}{s\,t} \quad \text{.......................(51)}$$

when the point, xy, is between the left support and g,

$$y = \frac{g\,x\,(s - x)}{s\,t} \quad \text{.......................(52)}$$

when the point, xy, is under the load, g; and

$$y = \frac{g\,m\,(s - x)}{s\,t} \quad \text{.......................(53)}$$

when the point, xy, is between g and the right support. These may be solved by the use of Fig. 15.

First, place the straight-edge on the tension scale and $(s—m)$ and mark the intercept on P.

Next, place the straight-edge from P to x and mark Q. Then, place the straight-edge from Q to g and mark R. Finally, placing the straight-edge from R to s, read the answer on y.

Thus,

$$y = \frac{2\,800 \times 100 \times 700}{1\,000 \times 60\,000} = 3.30 \text{ ft.}$$

The total deflection below the left-hand support is:

$$20 + 4.1 + 3.3 = 27.4 \text{ ft.}$$

The diagrams may also be used to find the deflection at a point, $x\,y$, for multi-loaded spans, thus (Fig. 16); let w = 3.5 lb.; t = 30 000 lb.; g = 1 800 lb.; a = 450 ft.; and m = 150 ft.

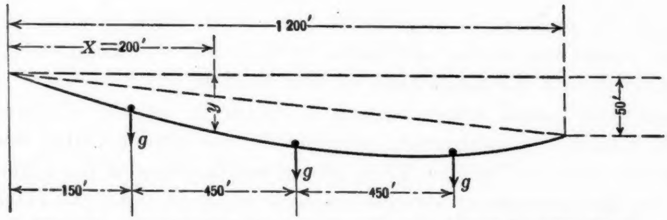

FIG. 16.—DEFLECTION AT ANY POINT ON A MULTI-LOADED SPAN.

(*a*) The slope factor is found from Fig. 15:

$$d = \frac{200 \times 50}{1\,200} = 8.3$$

Note since 50 does not appear on Fig. 15, use 500, but point off one place in the answer.

(*b*) The deflection of the cable is:

$$y = \frac{3.5 \times 200 \times 1\,000}{2 \times 30\,000} = 11.7 \text{ ft.}$$

(*c*) To find the deflections for loads treat each one as if it were separate. As the first load is to the left of xy, the following equations result:

By Equation (53):

$$y_1 = \frac{1\,800 \times 150 \times 1\,000}{1\,200 \times 30\,000} = 7.5$$

by Equation (52):

$$y_2 = \frac{1\,800 \times 200 \times 600}{1\,200 \times 30\,000} = 6.0; \; m = 150 + 450 = 600$$

by Equation (51):

$$y_3 = \frac{1\,800 \times 200 \times 150}{1\,200 \times 30\,000} = \underline{1.5}\; ; \; m = 150 + 450 + 450 = 1\,050$$

$$y_1 + y_2 + y_3 = 15.0$$

The diagrams may be used to find any of the unknown factors if the others are given. Thus, let $s = 1\,200$ ft.; $w = 3.5$ lb.; and $y = 12.5$ ft. at $x = \frac{s}{2}$; what is t?

Place the straight edge on $y = 12.5$ to $x = 600$, and mark Q; place it on Q to $w = 3.5$, and mark Q; and place it on P to $s - x = 600$, and read $t = 50\,500$ lb.

Anchored Spans.—A suspended cable hangs as a moment curve. The moment is the product of the deflection and tension. When weight-boxes are used the tension factor is made constant, and the deflection varies with the loading. It is often desirable to keep deflections within limits and have the tension vary with the loading. Such cases arise in the design of suspension bridges, cableways, and the long spans of aerial tramways. The ends of the cables of such spans are fastened to immovable anchorages; hence the term, anchored spans. To avoid injuring the cable, the loaded tension, T, is taken as the criterion for design. As it is not often practicable to erect the cable under load, the problem is to compute the empty cable erection tension, so that when loaded the tension developed in the cable will not exceed, but will be of approximately the magnitude of, the tension, T.

Cables, when placed under tension, increase in length. A part of this increment remains as permanent stretch, the remainder being due to the elastic properties of the cable. After several applications of the highest working tension, the permanent stretch becomes constant, and the elastic properties of the cable are not masked by it within this limit of tension. Therefore, anchored spans are calculated on the assumption that the cables will be erected and brought to a tension somewhat higher than the full working tension and then slackened to the empty tension.

For a span supporting a single load, such as a cableway, for instance, the loaded length, L, is given for every position of the load, by the formula,

$$L = s + \frac{G\,m}{2\,s\,T}(G + w\,s) + \frac{w^2\,s^3}{24\,T^2} + \frac{h^2}{2\,s} \quad \ldots\ldots\ldots\ldots (54)$$

As the maximum tension occurs when the load is at the middle of the span (see Equation (54)), the length of the empty span is,

$$l = s + \frac{w^2\,s^3}{24\,t^2} + \frac{h^2}{2\,s} \quad \ldots\ldots\ldots\ldots (55)$$

The change in tension, $T - t$, however, causes a shortening of the cable ($\varDelta\, L$). Then,

$$\varDelta\, L = \frac{(T - t)\, L}{A\, E} = (T - t)\, J \ldots\ldots\ldots\ldots\ldots\ldots (56)$$

Therefore,

$$L - \varDelta\, L = l, \text{ or } L = (T - t)\, J + s + \frac{w^2\, s^3}{24\, t^2} + \frac{h^2}{2\, s} \ldots\ldots\ldots\ldots (57)$$

$$J\, t + L - \left(T\, J + s + \frac{h^2}{2\, s}\right) = \frac{w^2\, s^3}{24\, t^2} \ldots\ldots\ldots\ldots\ldots\ldots (58)$$

a cubic equation having one real root.

Example.—Let $s = 1\,000$ ft.; $G = 2\,000$ lb.; $w = 4$ lb.; $T = 40\,000$ lb.; $h = 100$ ft.; $E = 20\,000\,000$ lb.; and $A = 1.1$ sq. in.; $t = ?$

$$L = 1\,000 + \frac{2\,000 \times 1\,000}{8 \times 40\,000^2}\,(2\,000 + 4\,000) + \frac{4^2 \times 1\,000^3}{24 \times 40\,000^2}$$

$$+ \frac{100^2}{2 \times 1\,000}\, 1\,000 + 0.9375 + 0.4167 + 5 = 1\,006.354 \text{ ft.}$$

$$\frac{L}{A\, E} = \frac{1\,006.354}{1.1 \times 2 \times 10^7} = 0.0000457 = J$$

$$T\, J = 40\,000 \times 0.0000457 = 1.828$$

$$L - \left(T\, J + s + \frac{h^2}{2\, s}\right) = 1\,006.354 - (1.828 + 1\,000 + 5) - 0.474$$

$$\frac{w^2\, s^3}{24} = \frac{4^2 \times 1\,000^3}{24} = 666\,666\,667$$

Then,

$$0.0000457\, t - 0.474 = \frac{666\,666\,667}{t^2}$$

Table 22 results from slide-rule computations, showing a succession of assumed values of t until the equation is satisfied (the values in the last two columns are equal).

TABLE 22.—COMPUTATIONS TO FIND VALUE OF t.

Values of t.	$0.0000457\, t$.	-0.474	$\frac{666\,666\,667}{t^2}$
30 000 lb.	1.371	0.897	0.741
29 000 "	1.325	0.851	0.794
28 000 "	1.280	0.806	0.852
28 400 "	1.308	0.834	0.830

The erection tension is, therefore, 28 400 lb. The center deflections of this span are:

Loaded	= 75	ft. below left support.
Empty	= 67.6	" " " "
Difference	= 7.4 ft.	

If the cables were non-elastic, this difference in deflection would be very much smaller than indicated.

Conversely: To find T if a load, G, is placed on the mid-point of an empty span:

$$l - \left(J\,t + s + \frac{h^2}{2\,s}\right) = \frac{G\,s\,(G + w\,s)}{8\,T^2} + \frac{w^2\,s^3}{24\,T^2} - J\,T \ldots\ldots (59)$$

Example.— s = 1 000 ft.; G = 2 000 lb.; w = 4 lb.; t = 28 400 lb.; h = 100 ft.; $E = 2 \times 10^7$; and A = 1.10 sq. in.; T = ?

$$l = 1\,000 + 0.827 + 5 = 1\,005.827 \text{ ft.}$$

$$J = 0.0000457$$

$$J\,t = 1.298$$

$$l - \left(J\,t + s + \frac{h^2}{2s}\right) = 1\ 005.827 - 1\ 006.298 = -\,0.471$$

$$\frac{G\,s\,(G + w\,s)}{8}\ \frac{2\ 000 \times 1\ 000 \times 6\ 000}{8} = 1\ 500\ 000\ 000$$

$$\frac{w^2\,s^3}{24} = \frac{4^2 \times 1\ 000^3}{24} = \frac{66\ 670\ 000}{2\ 166\ 700\ 000}$$

$$0.0000457\,T - 0.471 = \frac{2\ 166\ 700\ 000}{T^2}$$

TABLE 23.—COMPUTATIONS TO FIND VALUES OF T.

Values of T.	$0.0000457\,T$	-0.471	$\frac{2\ 166\ 700\ 000}{T^2}$
40 000	1.830	1.359	1.355

Therefore, the loaded tension will be 40 000 lb.

Aerial Dumping.—Cableways and aerial tramways are often used to accumulate stock or waste piles, in which case aerial dumping is required. To prevent the carrier leaving the cable, the weight of the load dumped should not exceed twice the weight of the empty carrier plus the weight of the cable of the span. In practice, it is best to make these weights equal, if unwelcome oscillations of the span are to be avoided.

If the load is not released instantly, then the minimum time interval for dumping it should exceed, in seconds, one-fourth the square root of the difference in deflection of the span before and after discharging the load. Thus, if the difference in deflection of the span when supporting the loaded and empty carrier is 4.82 ft., the time of dumping must exceed $\frac{1}{4}\sqrt{4.82}$, or 0.55 sec.

To protect roads, railways, or other points of danger, guard screens (see Fig. 7), suspended from anchored cables, are often installed. In order to design the structure properly, it becomes necessary to compute the instantaneous deflection of the screen due to the falling carrier.

The following solution of the problem is presented for the case of single spans with weighted or anchored cables.

Theory of Guard Screens.

Let the normal position of the guard screen be as shown in Fig. 17. The notation used in computing the formulas for guard screens is, as follows:

s = horizontal length of span.
h = difference in terminal elevation.
W = virtual weight of the span per foot. W is to be increased by sec α when $\alpha > 10°$.
y_0 = normal deflection below the chord.
y_1 = normal deflection increased 1 ft.
y = deflection at any instant.
x = increase in normal deflection at any instant.
t_0 = tension for normal deflection.
t_1 = tension for y_1 deflection.
T = tension for y deflection.
P_0 = resistance of screen to 1 ft. deflection.
P = resistance of screen to any deflection.
L_0 = length of cable (normal).
H = fall of carrier.
$G_0 = \dfrac{\text{weight of loaded carrier}}{\text{number of cables}}$.
G_1 = hypothetical load on screen for 1 ft. deflection.
G = hypothetical load on screen for any deflection.
$d\,t_1 = t_1 - t_0$.
$d\,t = T - t_0$.

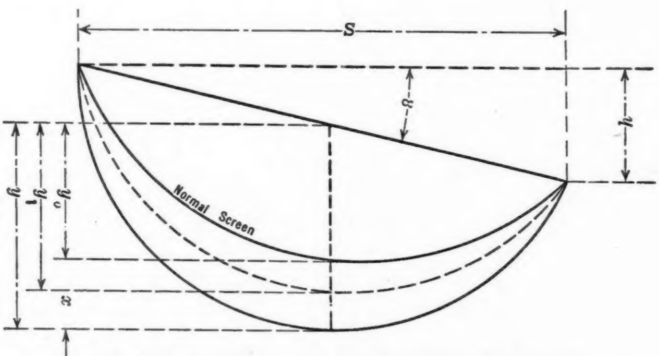

Fig. 17.—Design of a Single-Span Guard Screen.

$$y_0 = \frac{W\,s^2}{8\,t_0} \qquad \text{(60)}$$

$$y_1 = \frac{W\,s^2 + 2\,G_1\,s}{8\,(t_0 + d\,t_1)} = \frac{P_0\,s}{8\,t_1} \qquad \text{(61)}$$

$$y = \frac{W\,s^2 + 2\,G\,s}{8\,T} = \frac{P\,s}{8\,(t_0 + d\,t)}. \qquad \text{(62)}$$

$$y : y_1 = \frac{P\,s}{8\,(t_0 + d\,t)} : \frac{P_0\,s}{8\,(t_0 + d\,t_1)} \qquad \text{(63)}$$

But, $y = y_0 + x$:

$$P = \frac{P_0\, y\,(t_0 + d\,t)}{y_1\,(t_0 + d\,t_1)} = \frac{P_0\,(y_0 + x)(t_0 + d\,t)}{y_1\,(t_0 + d\,t_1)} = \frac{P_0\,(y_0\,t_0 + y_0\,d\,t + t_0\,x + x\,d\,t}{y_1\,(t_0 + d\,t_1)} \quad (64)$$

$$L_0 = s + \frac{8\,y_0^2}{3\,s},\ L = s + \frac{8\,y^2}{3\,s} \quad \ldots\ldots\ldots\ldots (65)$$

$$\lambda = L - L_0 = \frac{8\,(y^2 - y_0^2)}{3\,s} \quad \ldots\ldots\ldots\ldots (66)$$

$$d\,t = \frac{\lambda\,A\,E}{L_0} = \frac{8\,(2\,y_0\,x + x^2)\,A\,E}{3\,s\,L_0} = K\,(2\,y_0\,x + x^2) \quad \ldots\ldots (67)$$

Substitute in Equation (64):

$$P = \frac{P_0\,(y_0\,t_0 + 2\,K\,y_0^2\,x + K\,y_0\,x^2 + t_0\,x + 2\,K\,y_0\,x^2 + K\,x^3)}{y_1\,(t_0 + d\,t_1)} \quad \ldots\ldots (68)$$

$$v\,d\,v = p\,d\,x \quad \ldots\ldots\ldots\ldots (69)$$

Since the carrier falls a distance, H, it has a velocity C ft. per sec. when it strikes the screen. This velocity is brought to zero in the distance, x. To state acceleration, p,

$$p = (G_0 - P)\,\frac{g}{G_0} \quad \ldots\ldots\ldots\ldots (70)$$

$$\frac{1}{g}\,v\,d\,v = d\,x - \frac{P}{G_0}\,d\,x$$

Considering the downward direction as positive and integrating:

$$\frac{1}{g}\int_c^0 v\,d\,v = \int_0^x d\,x - \frac{P_0}{G_0\,y_1\,t_1}\int_0^x (y_0\,t_0 + 2\,K\,y_0^2\,x + K\,y_0\,x^2 + t_0\,x$$
$$+ 2\,K\,y_0\,x^2 + K\,x^3) \quad \ldots\ldots\ldots\ldots (71)$$

Let $\frac{P_0}{G_0\,y_1\,t_1} = B$:

$$+ H = -x + B\,y_0\,t_0\,x + B\,K\,y_0^2\,x^2 + \frac{B\,K\,y_0\,x^3}{3} + \frac{B\,t_0\,x^2}{2}$$
$$+ \frac{2\,B\,K\,y_0\,x^3}{3} + \frac{B\,K\,x^4}{4} \quad \ldots\ldots\ldots\ldots (72)$$

$$x^4 + 4\,y_0\,x^3 + 2\left[2\,y_0^2 + \frac{t_0}{K}\right]x^2 - \frac{4\,[B\,y_0\,t_0 - 1]}{K\,B}\,x - \frac{4\,H}{K\,B} = 0 \quad \ldots (73)$$

Weighted Span.—The resistance of the screen is proportional to its deflection. Tension is constant. (See Fig. 18.)

$$y_0 = \frac{W\,s^2}{8\,t} \quad \ldots\ldots\ldots\ldots (74)$$

$$y_1 = \frac{W\,s^2 + 2\,G_1\,s}{8\,t} = \frac{P_0\,s}{8\,t} \quad \ldots\ldots\ldots\ldots (75)$$

$$y = \frac{W\,s^2 + 2\,G\,s}{8\,t} = \frac{P\,s}{8\,t} \quad \ldots\ldots\ldots\ldots (76)$$

$$y : y_1 = \frac{P\,s}{8\,t} : \frac{P_0\,s}{8\,t}$$

But $y = y_0 + x$:

$$P = \frac{P_0\, y}{y_1}\,;\; d\,t = 0$$

Substitute in $v\,d\,v = p\,d\,x$:

$$v\,d\,v = \frac{(G - p)\,g}{G}\,d\,x \qquad (77)$$

or,

$$\frac{v\,d\,v}{g} = d\,x - \frac{P}{G}\,d\,x$$

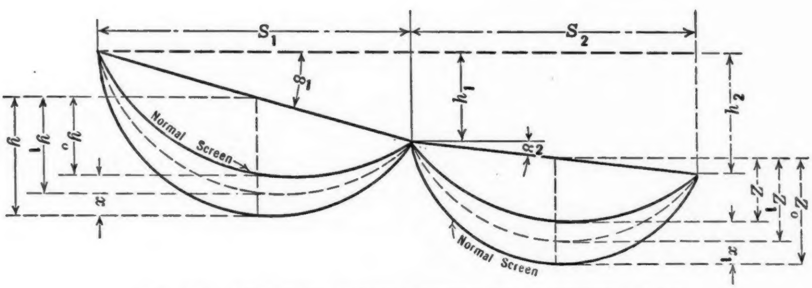

FIG. 18.—DESIGN OF GUARD SCREENS ON ADJOINING SPANS.

Integrating,

$$\frac{1}{g}\int_c^0 v\,d\,v = \int_0^x d\,x - \frac{P_0}{G\,y_0}\int x\,d\,x \qquad (78)$$

$$\frac{1}{g}\left[0 - \frac{C^2}{2}\right] = (x - 0) - \frac{P_0}{G\,y_0}\left[\frac{x^2}{2} - 0\right] \qquad (79)$$

$$-H = x - \frac{P_0}{G\,y_0}\cdot\frac{x^2}{2} \qquad (80)$$

$$x = \frac{G\,y_0}{P_0} \pm \frac{1}{2}\sqrt{\left(\frac{2\,G\,y_0}{P_0}\right)^2 + \frac{8\,H\,G\,y_0}{P_0}} \qquad (81)$$

Adjoining Spans.—(See Fig. 18):

$$y_0 = \frac{W_1\,s_1^{\,2}}{8\,t_0},\; Z_0 = \frac{W_2\,s_2^{\,2}}{8\,t_0}\,;\; y_1 - y_0 = 1\text{ ft.} \qquad (82)$$

$$y_1 = \frac{W_1\,s_1^{\,2} + 2\,G_1\,s_1}{8\,(t_0 + d\,t_1)}\,;\; Z_1 = \frac{W_2\,s_2^{\,2}}{8\,(t_0 + d\,t)} \qquad (83)$$

$$L_{y_1} = \frac{8\,y_1^{\,2}}{3\,s_1}\,;\; L_{Z_1} = \frac{8\,Z_1^{\,2}}{3\,s_2} \qquad (84)$$

Let b = length of back-stays. Then, the normal length is $L_{y0} + L_{Z_0} + b = L_0$. $L_0\,\lambda = B$.

$$L_{y_1} + L_{Z_1} + b = L_{y_0} + L_{Z_0} + b + d\,L_1 \qquad (85)$$

$$(L_{y_1} - L_{y_0}) = C\,;\; (L_{Z_1} - L_{Z_0}) = d = \frac{W_2^{\,2}\,s_2^{\,3}}{24\,(t^2 - t_0^{\,2})} \qquad (86)$$

therefore,

$$C + d - d\,L_1 = 0\,;\quad d\,L_1 = \lambda\,(L_{y_0} + L_{Z_0} + b)\,(t - t_0)$$

$$C + \frac{W_2^{\,2}\,s_2^{\,3}}{24\,(t^2 - t_0^{\,2})} - B\,(t - t_0) = 0 \quad \text{(87)}$$

This gives the solution for t, from which G_1 and P_0 can be computed.

$$d\,L = L - L_0 = \frac{8}{3\,s_1}\,(y^2 - y_0^{\,2}) + \frac{8}{3\,s_2}\,(Z^2 - Z_0^{\,2}) \quad \text{(88)}$$

But,

$$y = y_0 + x;\; Z = Z_0 - x_1$$

$$d\,t = \frac{d\,L\,A\,E}{L_0} = \frac{8}{3}\,\frac{(2\,y_0\,x + x^2)\,A\,E}{L_0} + \frac{8}{3}\,\frac{(-\,2\,Z_0\,x_1 + x_1^{\,2})\,A\,E}{L_0} \quad \text{(89)}$$

But,

$$x_1 = \frac{W_2\,s_2}{W_1\,s_1}\,x = J\,x$$

$$d\,t = K\,[2\,(y_0 + Z_0\,J)\,x + (1 + J^2)\,x^2] \quad \text{(90)}$$

$$P = \frac{P_0\,(y_0\,t_0 + y_0\,K\,[2\,(y_0 + Z_0\,J)\,x + (1 + J^2)\,x^2] + t_0\,x + x\,K\,[2\,(y_0 + Z_0\,J)\,x + (1 + J^2)\,x^2]}{y_1\,(t_0 + d\,t_1)} \quad \text{(91)}$$

$$v\,d\,v = p\,d\,x$$

Since the velocity of the falling carrier is brought to zero in the distance, x, to state acceleration, p,

$$p = (G_0 - P)\,\frac{g}{G_0} \quad \text{(92)}$$

$$\frac{1}{g}\,v\,d\,v = d\,x - \frac{P}{G_0}\,d\,x$$

Considering the downward direction as positive and integrating,

$$\frac{1}{g}\int_c^0 v\,d\,v = \int_0^t d\,x - \frac{P_0}{G_0\,y_1\,t_1}\int_0^x \{y_0\,t_0 + y_0\,K\,[2\,(y_0 + Z_0\,J)\,x + (1 + J^2)\,x^2\,] + t_0\,x + x\,K\,[2\,(y_0 + Z_0\,J)\,x + (1 + J^2)\,x^2]\} \quad \text{(93)}$$

Let $\dfrac{P_0}{G_0\,y_1\,t_1} = B$,

$$+\,H = -\,x + B\,y_0\,t_0\,x + B\left[\frac{t_0}{2} + y_0\,K\,(y_0 + Z_0\,J)\right]x^2 + \frac{B\,K\,[y_0 + y_0\,J^2 + 2_{y_0} + 2\,Z_0\,J]\,x^3}{3} + \frac{B\,K\,[1 + J^2]\,x^4}{4} \quad \text{(94)}$$

$$x^4 + \frac{4\,y_0\,[3 + J^2 - 2\,Z_0\,J]}{3\,[1 + J^2]}\,x^3 + \frac{4\left[\dfrac{t_0}{2} + y_0\,K\,(y_0 + Z_0\,J)\right]}{K\,[1 + J^2]}\,x^2 + \frac{4\,[B\,y_0\,t_0 - 1]}{B\,K\,[1 + J^2]}\,x - \frac{4\,H}{B\,K\,[1 + J^2]} = 0 \quad \text{(95)}$$

Anchored Cables.—

Example.—Let W = 10 lb. per ft.; t_0 = 5 000 lb.; y_0 = 10 ft.; G_0 = 1 000 lb.; H = 12 ft.; s = 200 ft.; A = 1.1 sq. in.; and $E = 20 \times 10^6$. Then:

$$L_0 = 200 + \frac{8 \times 10^2}{3 \times 200} = 201.33 \text{ ft.}; \; L_1 = 200 + \frac{8 \times 11^2}{3 \times 200} = 201.61 \text{ ft.}$$

$$\lambda = 201.61 - 201.33 = 0.28 \text{ ft.}$$

$$d\,t_1 = \frac{0.28 \times 1.1 \times 20 \times 10^6}{201.33} = 30\,600 \text{ lb.}$$

$$t_1 = 30\,600 + 5\,000 = 35\,600 \text{ lb.}$$

$$K = \frac{8 \times 1.1 \times 20 \times 10^6}{3 \times 200 \times 201.33} = 1\,460$$

$$P_0 = \frac{11 \times 8 \times 35\,600}{200} = 15\,700 \text{ lb.}$$

$$B = \frac{15\,700}{1\,000 \times 11 \times 35\,600} = 0.00004$$

$$x^4 + 40\,x^3 + 406.86\,x^2 - 68.4\,x - 825 = 0$$

$$x = 1.41 \text{ ft.}; \; \lambda = 0.41 \text{ ft.}; \; T = 49\,675 \text{ lb.}$$

Weighted Cables.—

$$t = \frac{10 \times 200^2}{8 \times 10} = 5\,000 \text{ lb.}$$

$$P_0 = \frac{40\,000 \times 11}{200} = 2\,200 \text{ lb.}$$

$$x = \frac{1\,000 \times 10}{2\,200} + \frac{1}{2}\sqrt{\frac{2 \times 1\,000 \times 10}{2\,200} + \frac{8 \times 12 \times 1\,000 \times 10}{2\,200}}$$
$$= 4.53 \quad + 5.6 \quad = 10.13 \text{ ft.}$$

*Friction.—*The coefficient of friction, k, of tramway carriages moving at speeds less than 700 ft. per min. were determined to be as shown in Table 24.

TABLE 24.—COEFFICIENT OF FRICTION OF TRAMWAY CARRIAGES.

Load, in pounds.	Wheel-base, in inches.	Journals.*	Coefficient of friction, k.
5 525	21.5	Ball-bearings	0.0045
1 535	21.5	" "	0.0031
2 525	13.75	Cast-steel wheels on bronze pins	0.0192
2 500	13.75	Babbitt bushings on steel pins	0.0158
2 500	13.75	Bronze bushings on steel pins	0.0163
535	13.75	Bronze bushings on steel pins	0.0122
1 240	11.875	Cast-steel wheels on bronze pins	0.0160
1 245	11.875	Ball-bearing	0.0108
255	11.875	Cast-steel wheels on bronze pins	0.0073
260	11.875	Ball-bearing	0.0083

* For roller bearings, multiply values for ball bearings by 1.5.

*Coasting Distance.—*It is often necessary to determine the coasting distance of a detached carrier.

Let, $\frac{G}{32.2}$ = mass of tramway carrier; G = weight, in pounds

p = acceleration, plus or minus

F = friction

$\frac{F}{G} = k$ = coefficient of friction

S = distance traveled

V = velocity

H = velocity head

T = time, in seconds

Then,

$$F = \frac{G}{32.3} p; \; p = 32.2\, k; \; S = \frac{V_1^2 - V_0^2}{2\, p} = \frac{0.0155\,(V_1^2 - V_0^2)}{k} \ldots. \;(96)$$

To coast to rest,

$$S = \frac{0.0155\; V^2}{k}, \text{ or } S = \frac{H}{k}.$$

Example.—If a carrier, running free upon a level rail, has an initial velocity of 8.33 ft. per sec. ($k = 0.0155$), how far will it coast?

$$S = \frac{0.0155 \times 8.33^2}{0.0155} = 69 \text{ ft.}$$

If the rail is not level, but has a slope equal to $\tan \theta$:

$$S = \frac{0.0155\; V^2}{k \pm \tan \theta} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (97)$$

Thus, in the previous example, if the rail grade is 1% upward:

$$S = \frac{0.0155 \times 8.33^2}{0.155 \times 0.01} = 41.9 \text{ ft.}$$

Example.—What upward grade should a rail have to stop a free running carrier moving 8.33 ft. per sec. ($k = 0.0155$), in a distance of 10 ft.?

$$\tan \theta = \frac{0.0155\; V^2}{S} - k\, S = \frac{0.0155 \times 8.33^2}{10} - 0.155 = 5.2\%$$

and the difference in elevation of the points on the rail 10 ft. apart is 0.52 ft. Should the time required to bring the carrier to rest be desired:

$$T = 0.25 \sqrt{\frac{S}{k \pm \tan \theta}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots. (98)$$

$$= 0.25 \sqrt{\frac{10}{0.0155 + 0.052}} = 1.73 \text{ sec.}$$

Tension in Traction Rope.—Let *A, B,* and *C,* Fig. 19, be loading, discharge, and curve rail stations, respectively, of an aerial tramway. It is desired to determine the traction rope tensions at Stations *A* and *C,* friction included. Motion is impending from Station *A* to Station *B,* loaded side. Let $T\, r$ = ten-

sion loaded side and $t\,r$ = tension empty side. The tensions at Station C for loaded and empty sides are, as follows:

$$T\,r = N_2\,G\,(\sin\theta_2 - k) + \frac{W}{2} \quad \text{(99)}$$

$$t\,r = N_2\,g\,(\sin\theta_2 + k) + \frac{W}{2} \quad \text{(100)}$$

in which, W is the weight of the traction rope weight-box, and k is the coefficient of friction.

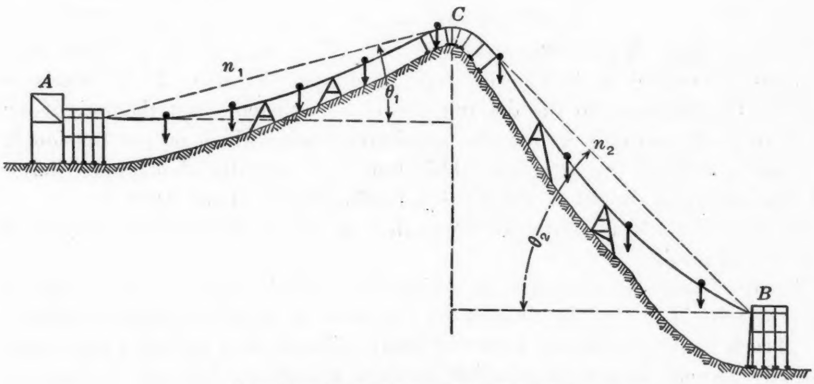

FIG. 19.—TENSION IN TRACTION ROPE.

The tension at Station A will be that at Station C increased or reduced by the loading of the Section A-C, depending on the elevation of Station A as compared with that of Station C. If Station A is lower than Station C, then,

$$T\,r = G\,[N_2\,(\sin\theta_2 - k) - N_1\,(\sin\theta_1 + k)] + \frac{W}{2} \quad \text{(101)}$$

$$t\,r = g\,[N_2\,(\sin\theta_2 + k) - N_1\,(\sin\theta_1 - k)] + \frac{W}{2} \quad \text{(102)}$$

$$\pm\,(T\,r - t\,r) = (G - g)\,(N_2\sin\theta_2 - N_1\sin\theta_1) - (G + g)\,(N_2\,k + N_1\,k) \quad \text{(103)}$$

As the relative values of $T\,r$ and $t\,r$ are not known, their difference may be plus or minus. Let this difference equal $\pm\,D$, then the horse power, $H\,P$, of the tramway may be estimated:

$$H\,P = \frac{\pm\,D\,V}{33\,000} - 2 \quad \text{(104)}$$

When the sign is plus, power is developed; when it is minus, power is required. The number, 2, represents the terminal friction reduced to horse power, and will be found sufficient for all stations of usual design.

To prevent the traction rope from slipping, $T\,r$ must have a proper value, $2\,T\,r_1$, which is,

$$2\,T\,r_1 = D\,(1 + K)$$

in which,

$$K = \frac{e^{f\pi n} + 1}{e^{f\pi n} - 1}$$

in which, f is the coefficient of friction of the rope on the sheave, and n is the number of half laps of the rope on the sheave.

For $n = 1$, K has the following values:

Greasy rope on an iron sheave	9.13
" " " a wood-filled sheave	4.62
" " " a rubber and leather-filled sheave	3.21
" " " a grip sheave	2.00

Traction Rope Weight-Box.—Let $W = 2\ (T\ r_1 - T\ r)$ be positive; then W equals the weight of the traction rope tension mechanism. If W is zero or negative, the adhesion to the driving sheave is sufficient, and the weight-box will be of such size only as may be required to maintain a proper tension in the traction rope at the attacher. This tension is usually about 1 000 lb., so that the minimum weight of the tension mechanism is about 2 000 lb.

The use of these formulas is illustrated in the mathematical analysis of the tramway profile.

Size of Track Cables.—The selection of a track cable for a carrier of known weight is one of the important problems of aerial tramway engineering. Track cables are to the tramway what rails are to a railway; they must be sturdy enough to give reasonable tonnage capacities, but not so large as to involve excessive first cost. The choice of track cables may be left to the results of experience, or immersed in the fog of theoretical speculation which surrounds the question of the bending stress in wire ropes. It is presumed that the determination of the bending stress in the wires of a rope has for its purpose the discovery of the reduction in strength of a wire rope due to bending. Those who have had occasion to make practical use of wire ropes by bending them around the small sheaves of wire-rope blocks, know that the loss in strength is an insignificant part of the amount it should be according to published tables based on bending stress. For instance, a 6 by 19¾-in. plow steel rope is reduced about 8% in strength when bent around a sheave 6 in. in diameter.

The velocity of the rope in passing around sheaves has a profound influence on its life and, for the same rope, sheaves of different diameters are required for equal service at different speeds. As aerial tramway velocities are usually less than 600 ft. per min., velocity factors may be eliminated from this discussion.

If the validity of the theory of flexure when applied to wire ropes is granted, it is easy to derive an expression for the curve of a track cable between the points of contact of the carriage wheels:

$$y = \frac{G}{t}\left[x - \frac{\sinh \sqrt{\frac{t}{E\ I}}\ x}{\sqrt{\frac{t}{E\ I}}\ \cosh \sqrt{\frac{t}{E\ I}}\ L}\right] \quad \ldots\ldots\ldots\ldots\ (105)$$

and,

$$\frac{d\,y}{d\,x} = \frac{G}{2\,t}\left[1 - \frac{\cosh \sqrt{\frac{t}{E\,I}}\,x}{\cosh \sqrt{\frac{t}{E\,I}}\,L}\right] \quad \text{(106)}$$

Where the origin is at the point of wheel contact and L is one-half the distance between wheel contacts, I is the moment of inertia of the rope. Therefore, the curve of cable bending is a function of the load, tension, wheel base, wheels, and the construction, as well as the metal of the cable. It is noted, from an examination of the data of lock-coil cable tests, that the modulus of elasticity is a function of the tension, which influences the amount of internal friction, and thus determines the behavior of the cable when loaded. Fig. 20 illustrates three complete cycles of progressive and retrogressive loading of a 1-in. lock-coil cable. It is apparent that the stress-strain curve of restitution does not coincide with the curve of deformation. After the permanent stretch has been eliminated, a closed foot-pound diagram results, from which the magnitude of the internal friction can be estimated in a manner similar to the mean effective pressure from an engine indicator card. Table 25 gives the internal friction of several lock-coil cables under sufficient tension to follow Hooke's law.

TABLE 25.—Internal Friction of Lock-Coil Cables.

Diameter, in inches.	Internal friction, in pounds.
7/8	4 900
1	7 700
1 1/8	8 800
1 1/4	9 700
1 1/2	13 100
1 5/8	14 800

Since the internal friction is due to the pressure exerted by the helical windings, and varies with the tension, it is clear that the cable resistance to bending increases with the tension until it takes on the semblance of a homogeneous bar, although it is noted that the modulus of elasticity does not reach the values of bars. Table 26 gives the value of the modulus of elasticity of various ropes under tensions equal to one-fourth the ultimate strength.

TABLE 26.—Moduli of Elasticity of Various Ropes.

Kind.	Size, in inches.	Values of E, in pounds.
Lock-coil, lock-wire, and smooth coil	1 1/4 and smaller	23 000 000
Lock-coil, lock-wire, and smooth coil, larger than	1 1/4	20 000 000
6 by 7 crucible	3/4	12 000 000
6 by 19 "	"	10 000 000
6 by 37 "	"	10 000 000
8 by 19 "	"	6 000 000
6 by 37 plow	2 1/4	15 000 000

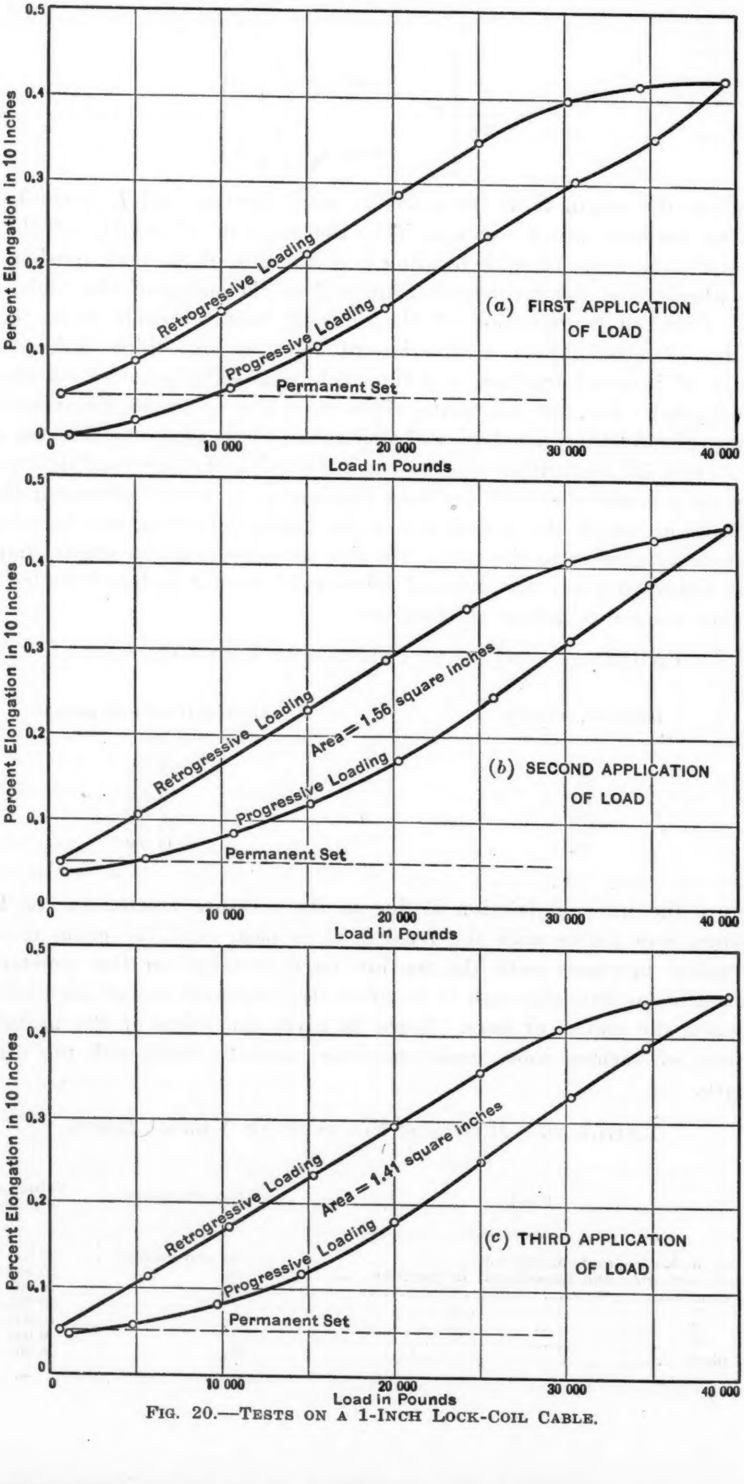

FIG. 20.—TESTS ON A 1-INCH LOCK-COIL CABLE.

For tensions less than one-quarter the ultimate, the following values of E (Table 27) are given for a 1-in. lock-coil cable.

TABLE 27.—MODULI OF ELASTICITY FOR LOCK-COIL CABLE.

Tension, in pounds.	Values of E, in pounds.
0	1 500 000
5 000	2 400 000
7 500	3 400 000
10 000	5 000 000
12 500	7 750 000
15 000	9 250 000
17 500	12 000 000
20 000	15 300 000
25 000	23 000 000

Having ascertained the value of the modulus of elasticity with regard to tension, the value of the stress set up in the outermost wire of the cable due to bending, may be computed approximately. It can be shown that the curve of the bent track cable does not differ greatly from a circular arc, and if the intersection angle of the tangents is Δ, the radius of curvature, R,″ will be,

$$R'' = \frac{W}{2 \times \sin \frac{\Delta}{2}} \qquad (107)$$

Table 28 gives the value of R for various chords and intersection angles for a 1-in. lock-coil cable.

TABLE 28.—VALUES OF R FOR VARIOUS CHORDS AND INTERSECTION ANGLES.

Δ, in degrees.	$\frac{G}{t}$.	Chord, in inches.	Radius, in feet.	Remarks.
1	0.0175	11.96	57	
2	0.0349	12.04	28.7	
3	0.0524	12.12	19.2	
4	0.0700	12.20	14.50	Wheel base. W = 11.875 in.
5	0.0875	12.28	11.7	Diameter of wheel = 8.25 in.
6	0.1051	12.36	9.9	
7	0.1228	12.44	8.5	
8	0.1405	12.52	7.5	

Having the radius, the bending stress, B, may be computed from the formula,

$$B = \frac{E\,d}{2\,R} \qquad (108)$$

in which, d is the diameter of the rope or cable, and R is the radius of the cable curve, in inches.

Tables 29 to 37 give the computed bending and direct stresses for several different sizes of lock-coil track cables. It will be noted that the sum of these stresses changes rapidly with the tension. This leads to the conclusion that their sum is a minimum for a given load and a variable

TABLE 29.—STRESSES IN $\frac{7}{8}$-INCH LOCK-COIL CABLE, OX CARRIAGE.*

(p = stress, in pounds per square inch, due to bending; E = bending modulus of elasticity for given tension; d = diameter of rope; R = radius of curvature of rope; $W = 11\frac{7}{8}$ in.; $r = 4\frac{1}{8}$ in.; $p = \frac{E\,d}{2\,R}$; area = 491 sq. in.)

Total tension in cable, in pounds	0.0	2 500	5 000	7 500	10 000	12 500	15 000	17 500	20 000	22 500	25 000
Tension in cable, in pounds per square inch	.....	5 100	10 200	15 300	20 400	25 500	30 600	35 600	40 700	45 800	50 900
E, in millions of pounds	1.0	1.6	3.4	6.8	10.5	16.0	22.5	22.5	22.5	22.5	22.5
Angle under load on O X carriage, in degrees.	STRESS IN OUTER WIRES DUE TO BENDING, IN POUNDS PER SQUARE INCH.										
1	640	1 025	2 175	4 350	6 725	10 250	14 400	14 400	14 400	14 400	14 400
		6 125	**12 375**	**19 650**	**27 125**	**35 750**	**45 000**	**50 000**	**55 100**	**60 200**	**65 300**
2	1 270	2 035	4 320	8 650	13 850	20 350	28 600	28 600	28 600	28 600	28 600
		7 135	**14 520**	**23 950**	**33 750**	**45 850**	**59 200**	**64 200**	**69 300**	**74 400**	**79 500**
3	1 890	3 020	6 425	12 850	19 800	30 200	42 600	42 600	42 600	42 600	42 600
		8 120	**16 625**	**28 150**	**40 200**	**55 700**	**73 200**	**78 200**	**83 300**	**88 400**	**93 500**
4	2 500	4 000	8 500	17 000	26 250	40 000	56 250	56 250	56 250	56 250	56 250
		9 100	**18 700**	**32 300**	**46 650**	**65 500**	**86 850**	**91 850**	**96 950**	**102 050**	**107 150**
5	3 100	4 960	10 540	21 100	32 600	49 600	69 750	69 750	69 750	69 750	69 750
		10 060	**20 740**	**36 400**	**53 000**	**75 100**	**100 350**	**105 350**	**110 450**	**114 550**	**120 650**
6	3 700	5 920	12 600	25 200	38 800	59 200	82 300	82 300	82 300	....	....
		11 020	**22 800**	**40 700**	**59 200**	**84 700**	**112 900**	**117 900**	**123 000**	....	....
7	4 280	6 850	14 550	29 100	45 000	68 500	96 300				
		11 950	**24 750**	**44 400**	**65 400**	**94 000**	**126 900**				
8	4 860	7 800	16 525	34 050	51 100	78 000	....				
		12 800	**26 725**	**49 350**	**71 500**	**103 500**	....				
10	6 000	9 600	20 400	40 800	63 000	96 000	....				
		14 700	**30 600**	**56 100**	**83 400**	**121 500**	....				
12	7 110	11 380	24 200	48 400	74 700	....	....				
		16 480	**34 400**	**63 700**	**95 100**	....	....				
14	8 200	15 100	27 850	55 700	86 200	....	....				
		20 200	**38 050**	**71 000**	**100 600**	....	....				
Breaking radius, in inches, 115 000 lb. per sq. in. ultimate.	3.8	6.1	12.9	25.8	40	61	86				

* Light and heavy figures indicate bending and total stresses, respectively; values below heavy line exceed 115 000 lb. per sq. in.

TABLE 30.—STRESSES IN 1-INCH LOCK-COIL CABLE, OX CARRIAGE.*

(p = stress, in pounds per square inch, due to bending; E = bending modulus of elasticity for given tension; d = diameter of rope; R = radius of curvature of rope; $p = \frac{E\,d}{2\,R}$; area = 662 sq. in.)

Total tension in cable, in pounds	0.0	5 000	7 500	10 000	12 500	15 000	17 500	20 000	25 000	30 000	35 000
Tension in cable, in pounds per square inch		7 550	11 300	15 100	18 900	22 600	26 400	30 200	37 800	45 300	52 800
E, in millions of pounds	1.5	2.4	3.4	5	7.75	9.25	12	15,3	23	23	23
Angle under load on O X carriage, in degrees.	**Stress in Outer Wires Due to Bending, in Pounds per Square Inch.**										
1	1 095	1 750	2 480	3 650	5 660	6 750	8 750	11 150	16 800	16 800	16 800
		9 300	**13 780**	**18 750**	**24 560**	**29 350**	**35 150**	**41 350**	**54 600**	**62 100**	**69 600**
2	2 175	8 480	4 930	7 250	11 300	13 400	17 400	22 200	33 400	33 400	33 400
		11 030	**16 230**	**22 350**	**30 200**	**36 000**	**43 800**	**52 400**	**71 200**	**78 700**	**86 200**
3	8 240	5 190	7 350	10 800	16 750	19 900	25 900	33 000	49 700	49 700	49 700
		12 740	**18 650**	**25 900**	**35 650**	**42 500**	**52 300**	**63 200**	**87 500**	**95 000**	**102 500**
4	4 290	6 870	9 750	14 300	22 200	26 400	34 400	48 800	65 800	65 800	65 800
		14 420	**21 050**	**29 400**	**41 100**	**49 000**	**60 800**	**74 000**	**101 100**	**111 100**	**118 600**
5	5 330	8 540	12 100	17 800	27 600	32 800	42 700	54 400	81 800	81 800	....
		16 090	**23 400**	**32 900**	**46 500**	**55 400**	**69 100**	**84 600**	**119 600**	**127 100**	....
6	6 350	10 200	14 400	21 200	32 800	39 100	50 800	64 800	97 500	....	....
		17 750	**25 700**	**36 300**	**51 700**	**61 700**	**77 200**	**95 000**	**135 300**	....	....
7	7 360	11 800	16 650	24 600	38 000	45 400	58 900	75 100	....	....	....
		19 350	**27 950**	**39 700**	**56 900**	**68 000**	**85 300**	**105 300**	....	....	....
8	8 350	13 400	18 900	27 800	43 200	51 500	66 800	85 100	....	....	....
		20 950	**30 200**	**42 900**	**62 100**	**74 100**	**93 200**	**115 300**	....	....	....
10	10 300	16 500	23 100	34 300	53 200	63 500	82 500				
		24 050	**34 400**	**49 400**	**72 100**	**86 100**	**108 900**				
12	12 200	19 500	27 600	40 700	63 000	75 100	97 600				
		27 050	**38 900**	**55 800**	**81 900**	**97 700**	**124 000**				
14	14 060	22 500	31 800	47 000	72 600	86 700	......				
		30 050	**43 100**	**62 100**	**91 500**	**109 360**	......				
Breaking radius, in inches, 115 000 lb. per sq. in. ultimate	6.53	10.43	14.75	21.75	33.75	40	52				

* Light and heavy figures indicate bending and total stresses, respectively; values below heavy line exceed 115 000 lb. per sq. in.

TABLE 31.—STRESSES IN $1\frac{1}{8}$-INCH LOCK-COIL CABLE, OX CARRIAGE.*

(p = stress, in pounds per square inch, due to bending; E = bending modulus of elasticity for given tension; d = diameter of rope; R = radius of curvature of rope; $p = \frac{E\,d}{2\,R}$; area = 842 sq. in.)

Total tension in cable, in pounds	0.0	2 500	5 000	7 500	10 000	12 500	15 000	17 500	20 000	25 000	30 000	35 000
Tension in cable, in pounds per square inch	.....	2 970	5 940	8 900	11 900	14 850	17 800	20 800	23 800	29 700	35 600	40 500
E, in millions of pounds	1.7	2.8	6.3	10.8	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0
Angle under load O X carriage, in degrees.	STRESS IN OUTER WIRES DUE TO BENDING, IN POUNDS PER SQUARE INCH.											
	1 400	2 300	5 200	8 900	16 350	16 350	16 350	16 350	16 350	16 350	16 350	16 350
1		**5 270**	**11 140**	**17 800**	**28 250**	**31 200**	**34 150**	**37 100**	**40 100**	**46 000**	**51 900**	**56 850**
	2 770	4 670	10 250	17 600	32 600	32 600	32 600	32 600	32 600	32 600	32 600	32 600
2		**7 640**	**16 190**	**26 500**	**44 500**	**47 450**	**50 400**	**53 400**	**56 400**	**62 300**	**68 200**	**73 100**
	4 130	6 800	15 300	26 200	48 600	48 600	48 600	48 600	48 600	48 600	48 600	48 600
3		**9 770**	**21 240**	**35 100**	**60 500**	**63 450**	**66 400**	**69 400**	**72 400**	**78 300**	**84 200**	**89 100**
	5 465	9 000	20 600	34 700	64 300	64 300	64 300	64 300	64 300	64 300	64 300	64 300
4		**11 970**	**26 540**	**43 600**	**76 200**	**79 150**	**82 100**	**85 100**	**88 100**	**94 000**	**99 900**	**104 800**
	6 780	11 150	25 100	43 100	79 800	79 800	79 800	79 800	79 800	79 800	79 800	79 800
5		**14 120**	**31 040**	**52 000**	**91 700**	**94 650**	**97 600**	**100 600**	**103 600**	**109 500**	**115 400**	**120 300**
	8 100	13 300	30 000	51 500	95 400	95 400	95 400	95 400	95 400	95 400		
6		**16 270**	**35 940**	**60 400**	**107 300**	**110 250**	**113 200**	**116 200**	**119 200**	**125 100**	....	....
	9 400	15 460	34 800	59 700	110 600	....	....					
7		**18 430**	**40 740**	**68 600**	**122 500**	....	....					
	10 600	17 450	39 300	67 300	....	....	....					
8		**20 420**	**45 240**	**76 200**	....	....	....					
	13 100	21 600	48 500	83 200	....	....	....					
10		**24 570**	**54 440**	**92 100**	....	....	....					
	15 550	25 600	57 600	98 700	....	....	....					
12		**28 570**	**63 540**	**107 600**	....	....	....					
	18 000	29 600	66 700	104 300	....	....	....					
14		**32 570**	**72 640**	**113 200**	....	....	....					
Breaking radius, in inches, 115 000 lbs. per sq. in., ultimate	8.33	13.7	30.8	53.0	98.0	....	....					

* Light and heavy figures indicate bending and total stresses, respectively; values below heavy line exceed 115 000 lb. per sq. in.

TABLE 32.—STRESSES IN $1\frac{1}{8}$-INCH LOCK-COIL CABLE, O X X CARRIAGE.*

(p = stress, in pounds per square inch, due to bending ; E = bending modulus of elasticity for given tension ; d = diameter of rope ; R = radius of curvature of rope ; $p = \frac{E\,d}{2\,R}$; area = 842 sq. in.)

Total tension in cable, in pounds	6.0	2 500	5 000	7 500	10 000	12 500	15 000	17 500	20 000	25 000	30 000	35 000
Tension in cable, in pounds per square inch	....	2 970	5 940	8 900	11 900	14 850	17 800	20 800	23 800	29 700	35 600	40 500
E, in millions of pounds	1.7	2.8	6.3	10.8	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0
Angle under load on O X X carriage, in degrees.	STRESS IN OUTER WIRES DUE TO BENDING, IN POUNDS PER SQUARE INCH.											
1	1 210	1 975	4 500	7 680	14 250	14 250	14 250	14 250	14 250	14 250	14 250	14 250
		4 945	**10 440**	**16 580**	**26 150**	**29 100**	**32 050**	**35 050**	**38 050**	**43 950**	**49 850**	**54 750**
2	2 420	3 985	8 970	15 350	28 500	28 500	28 500	28 500	28 500	28 500	28 500	28 500
		6 955	**14 910**	**24 250**	**40 400**	**43 350**	**46 300**	**49 300**	**52 300**	**58 200**	**64 100**	**69 000**
3	3 640	6 000	13 500	22 150	41 700	41 700	41 700	41 700	41 700	41 700	41 700	41 700
		8 970	**19 440**	**31 050**	**53 600**	**56 550**	**59 500**	**62 500**	**65 500**	**71 400**	**77 300**	**82 200**
4	4 840	7 980	17 900	30 800	57 000	57 000	57 000	57 000	57 000	57 000	57 000	57 000
		10 950	**23 840**	**39 700**	**68 900**	**71 850**	**74 800**	**77 800**	**80 800**	**86 800**	**92 600**	**97 500**
5	6 050	9 960	22 400	38 400	71 200	71 200	71 200	71 200	71 200	71 200	71 200	71 200
		12 930	**28 340**	**47 300**	**83 100**	**86 050**	**89 000**	**92 000**	**95 000**	**100 900**	**106 800**	**111 700**
6	7 250	11 920	26 900	46 000	85 300	85 300	85 300	85 300	85 800	85 300	85 800	......
		14 890	**32 840**	**54 900**	**97 200**	**100 150**	**103 100**	**106 100**	**109 100**	**115 000**	**120 900**	......
7	8 430	13 850	31 300	53 500	99 300	99 300	99 300					
		16 820	**37 240**	**62 400**	**111 200**	**114 150**	**117 100**					
8	9 660	15 900	35 800	61 400	113 500	......	......					
		18 870	**41 740**	**70 300**	**124 400**	......	......					
10	12 030	19 850	44 750	76 500	......	......	......					
		22 820	**50 690**	**85 400**	......	......	......					
12	14 380	23 700	53 300	91 400	......	......	......					
		26 670	**59 240**	**100 300**	......	......	......					
14	16 300	26 800	60 400	103 600	......	......	......					
		29 770	**66 340**	**112 500**	......	......	......					
Breaking radius, in inches, 115 000 lb. per sq. in., ultimate	8.33	13.7	30.8	53.0	98.0	......	......					

* Light and heavy figures indicate bending and total stresses, respectively ; values below heavy line exceed 115 000 lb. per sq. in.

TABLE 33.—STRESSES IN $1\frac{1}{4}$-INCH LOCK-COIL CABLE, O X CARRIAGE.*

(p = stress, in pounds per square inch, due to bending; E = bending modulus of elasticity for given tension; d = diameter of rope; R = radius of curvature of rope; $p = \frac{E\,d}{2\,R}$; area = 1.07 sq. in.)

Total tension in cable, in pounds	0.0	5 000	10 000	15 000	20 000	25 000	30 000	35 000	40 000
Tension in cable, in pounds per square inch	....	4 670	9 340	14 000	18 700	23 300	28 000	32 700	37 300
E, in millions of pounds	1.9	2.7	5.3	9.5	15.3	23.0	23.0	23.0	23.0
Angle under load on O X carriage, in degrees.	STRESS IN OUTER WIRES DUE TO BENDING, IN POUNDS PER SQUARE INCH.								
	1 735	2 470	4 850	8 650	14 000	21 000	21 000	21 000	21 000
1		**7 140**	**13 190**	**22 650**	**32 700**	**44 300**	**49 000**	**53 700**	**58 300**
	3 450	4 900	9 625	17 250	27 800	41 800	41 800	41 800	41 800
2		**9 570**	**18 965**	**31 250**	**46 500**	**65 100**	**69 800**	**74 500**	**79 100**
	5 130	7 300	14 300	25 600	41 400	62 200	62 200	62 200	62 200
3		**11 970**	**23 640**	**39 600**	**60 100**	**85 500**	**90 200**	**94 900**	**99 500**
	6 790	9 650	18 900	33 900	54 700	82 200	82 200	82 200	82 200
4		**14 320**	**28 240**	**47 900**	**73 400**	**105 500**	**110 200**	**114 900**	**119 500**
	8 420	12 000	28 500	42 200	67 800	102 000	....	....	....
5		**16 670**	**32 840**	**56 200**	**86 500**	**125 300**	....	....	....
	10 060	14 300	28 000	50 000	81 000	....	....	....	....
6		**18 970**	**37 340**	**64 000**	**99 700**	....	....	....	....
	11 650	16 550	32 500	58 300	94 000	....	....		
7		**21 220**	**41 840**	**72 400**	**112 700**	....	....		
	13 230	18 800	37 000	66 800	106 600	....	....		
8		**23 470**	**46 340**	**80 800**	**125 300**	....	....		
	16 320	23 200	45 600	81 600	....	....	....		
10		**27 870**	**54 940**	**95 600**	....	....	....		
	19 330	27 500	54 000	96 500	....	....	....		
12		**32 170**	**63 340**	**110 500**	....	....	....		
	22 250	31 600	62 800	112 500	....	....	....		
14		**36 270**	**72 140**	**126 500**	....	....	....		
Breaking radius, in inches, 115 000 lb. per sq. in. ultimate	10.33	14.625	28.75	50.0	84.0	125.0	....		

* Light and heavy figures indicate bending and total stresses, respectively; values below heavy line exceed 115 000 lb. per sq. in.

TABLE 34.—STRESSES IN $1\frac{1}{4}$-INCH LOCK-COIL CABLE, O X X CARRIAGE.*

(p = stress, in pounds per square inch, due to bending; E = bending modulus of elasticity for given tension; d = diameter of rope; R = radius of curvature of rope; $p = \frac{E\,d}{2\,R}$; area = 1.07 sq. in.)

Total tension in cable, in pounds	0.0	5 000	10 000	15 000	20 000	25 000	30 000	35 000	40 000	45 000
Tension in cable, in pounds per square inch		4 670	9 340	14 000	18 700	23 300	28 000	32 700	37 300	42 000
E, in millions of pounds	1.9	2.7	5.3	9.5	15.3	23.0	23.0	23.0	23.0	23.0
Angle under load on O X X carriage, in degrees.	STRESS IN OUTER WIRES DUE TO BENDING, IN POUNDS PER SQUARE INCH.									
1	1 500	2 180	4 180	7 500	12 100	18 200	18 200	18 200	18 200	18 200
		6 800	**13 520**	**21 500**	**30 800**	**41 500**	**46 200**	**50 900**	**55 500**	**60 200**
2	3 000	4 270	8 370	15 000	24 200	36 400	36 400	36 400	36 400	36 400
		8 940	**17 710**	**29 000**	**42 900**	**59 700**	**64 400**	**69 100**	**73 700**	**76 400**
3	4 500	6 400	12 550	22 500	36 200	54 500	54 500	54 500	54 500	54 500
		11 070	**21 890**	**36 500**	**54 900**	**77 800**	**82 500**	**87 200**	**91 800**	**96 500**
4	6 000	8 530	16 700	30 000	48 300	72 700	72 200	72 200	72 200	72 200
		13 200	**26 040**	**44 000**	**67 000**	**96 000**	**100 700**	**105 400**	**110 000**	**114 700**
5	7 500	10 650	20 900	37 500	60 400	91 000	91 000	91 000	....	....
		15 320	**30 240**	**51 500**	**79 100**	**113 300**	**119 000**	**123 700**	....	....
6	9 000	12 780	25 100	45 000	72 500	109 000	....	....	....	....
		17 450	**34 440**	**59 000**	**91 200**	**132 300**	....	....	....	....
7	10 500	14 900	29 800	52 500	84 500	....	....			
		19 540	**38 640**	**66 500**	**103 200**	....	....			
8	11 950	17 000	33 400	59 750	95 800	....	....			
		21 670	**42 740**	**73 750**	**114 500**	....	....			
10	15 000	21 300	41 800	75 000	121 000	....	....			
		25 970	**51 140**	**89 000**	**139 700**	....	....			
12	17 900	25 400	50 000	89 500	....	....	....			
		30 070	**59 340**	**103 500**	....	....	....			
14	20 800	29 600	58 000	104 000	....	....	....			
		34 270	**67 240**	**118 000**	....	....	....			
Breaking radius, in inches, 115 000 lb. per sq. in. ultimate	10.33	14.625	28.75	50.0	84.0	125.0	....			

* Light and heavy figures indicate bending and total stresses, respectively.

TABLE 35.—STRESSES IN $1\frac{3}{8}$-INCH LOCK-COIL CABLE, O X X CARRIAGE.*

(p = stress, in pounds, per square inch, due to bending; E = bending modulus of elasticity for given tension; d = diameter of rope; R = radius of curvature of rope; $p = \frac{E\,d}{2\,R}$, area = 1.266 sq. in.)

Total tension in cable, in pounds	0.0	5 000	10 000	15 000	20 000	25 000	27 500	30 000	35 000	40 000	45 000	50 000
Tension in cable, in pounds per square inch		3 950	7 900	11 850	15 800	19 750	21 700	23 700	27 600	31 600	35 500	39 500
E, in millions of pounds	2.2	2.8	4.6	7.6	11.8	17.2	20.0	20.0	20.0	20.0	20.0	20.0
Angle under load O X X carriage, in degrees.	STRESS IN OUTER WIRES DUE TO BENDING, IN POUNDS PER SQUARE INCH.											
1	1 920	2 440	4 020	6 630	10 300	15 000	17 500	17 500	17 500	17 500	17 500	17 500
		6 390	**11 920**	**18 480**	**26 100**	**34 750**	**39 200**	**41 200**	**45 100**	**49 100**	**53 000**	**57 000**
2	3 840	4 900	8 030	13 250	20 600	30 000	35 000	35 000	35 000	35 000	35 000	35 000
		8 850	**15 930**	**25 100**	**36 400**	**49 750**	**56 700**	**58 700**	**62 600**	**66 600**	**70 500**	**74 500**
3	5 750	7 320	12 000	19 800	30 800	45 000	52 400	52 400	52 400	52 400	52 400	52 400
		11 270	**19 900**	**31 650**	**46 600**	**64 750**	**74 100**	**76 100**	**80 000**	**84 000**	**89 900**	**91 900**
4	7 650	9 750	16 000	26 400	41 000	60 000	69 600	69 600	69 600	69 600	69 600	69 600
		13 700	**23 900**	**38 250**	**56 800**	**79 750**	**91 300**	**93 300**	**97 200**	**101 200**	**105 100**	**109 100**
5	9 550	12 200	20 000	33 000	51 200	74 800	88 000	88 000	88 000	88 000	....	....
		16 150	**27 900**	**44 850**	**67 000**	**94 550**	**109 700**	**111 700**	**115 600**	**119 600**	....	....
6	11 450	14 650	23 900	39 800	61 500	89 500	104 000	....	....			
		18 600	**31 800**	**51 650**	**77 300**	**109 250**	**124 700**	....	....			
7	13 350	17 000	27 900	46 200	71 700	104 500	....					
		20 950	**35 800**	**58 050**	**87 600**	**124 250**	....					
8	15 250	19 400	31 900	52 700	81 800	....	....					
		23 350	**39 800**	**64 550**	**97 600**	....	....					
10	19 000	24 200	39 700	65 700	102 000	....	....					
		28 150	**47 600**	**77 550**	**117 800**	....	....					
12	22 600	28 800	47 250	78 000	....	....	....					
		32 750	**55 150**	**89 850**	....	....	....					
14	26 400	34 900	55 200	91 000	....	....	....					
		38 850	**63 100**	**102 850**	....	....	....					
Breaking radius, in inches, 115 000 lb. per sq. in., ultimate	13.125	16.75	27.5	45.5	71.0	102.5	120.0					

* Light and heavy figures indicate bending and total stresses, respectively; values below heavy line exceed 115 000 lb. per sq. in.

TABLE 36.—STRESSES IN $1\frac{1}{2}$-INCH LOCK-COIL CABLE, O X X CARRIAGE.*

(p = stress, in pounds per square inch, due to bending; E = bending modulus of elasticity for given tension; d = diameter of rope; R = radius of curvature of rope; $p = \frac{E\,d}{2\,R}$; area = 1.464 sq. in.)

Total tension in cable, in pounds	0.0	5 000	10 000	15 000	20 000	25 000	30 000	35 000	40 000	45 000	50 000	55 000	60 000
Tension in cable, in pounds per square inch		3 420	6 830	10 250	13 650	17 000	20 500	23 900	27 300	30 700	34 100	37 500	41 000
E, in millions of pounds	2.5	3.0	4.5	6.9	10.3	14.6	20.0	20.0	20.0	20.0	20.0	20.0	20.0
Angle under load O X X carriage, in degrees.	STRESS IN OUTER WIRES DUE TO BENDING, IN POUNDS PER SQUARE INCH.												
1	2 880	2 860	4 280	6 560	9 820	13 900	19 000	19 000	19 000	19 000	19 000	19 000	19 000
		6 280	**11 110**	**16 810**	**23 470**	**30 900**	**39 500**	**42 900**	**46 300**	**49 700**	**53 100**	**56 500**	**60 000**
2	4 750	5 700	8 550	13 100	19 550	27 750	38 000	38 000	38 000	38 000	38 000	38 000	38 000
		9 120	**15 380**	**23 350**	**33 200**	**44 750**	**58 500**	**61 900**	**65 300**	**68 700**	**72 100**	**75 500**	**79 000**
3	7 150	8 570	12 900	19 700	29 450	41 750	57 200	57 200	57 200	57 200	57 200	57 200	57 200
		11 990	**19 730**	**29 950**	**43 100**	**58 750**	**77 700**	**81 100**	**84 400**	**87 900**	**91 300**	**94 700**	**98 200**
4	9 500	11 400	17 100	26 200	39 150	55 500	76 000	76 000	76 000	76 000	76 000	76 000	76 000
		14 820	**23 930**	**36 450**	**52 800**	**72 500**	**96 500**	**99 900**	**103 300**	**106 700**	**110 100**	**113 500**	**117 000**
5	11 850	14 200	21 300	32 700	48 800	69 300	95 000	95 000	95 000	....			
		17 620	**28 130**	**42 950**	**62 450**	**86 300**	**115 500**	**118 900**	**122 300**	....			
6	14 200	17 000	25 600	39 200	58 500	83 000	....	....	....	....			
		20 420	**32 430**	**49 450**	**72 450**	**100 000**	....	....	....	....			
7	16 500	19 800	29 700	45 500	68 000	96 500	....						
		23 220	**36 530**	**55 750**	**81 650**	**113 500**	....						
8	18 900	22 850	34 000	52 200	78 000	....	....						
		26 270	**40 830**	**62 450**	**91 650**	....	....						
10	23 600	28 300	42 500	65 000	97 300	....	....						
		31 720	**49 330**	**75 250**	**110 950**	....	....						
12	28 200	33 800	50 800	77 800	....	....	....						
		37 220	**57 630**	**88 050**	....	....	....						
14	33 000	39 600	59 500	91 000	....	....	....						
		43 020	**66 330**	**101 250**	....	....	....						
Breaking radius, in inches, 115 000 lb. per sq. in., ultimate	16.3	19.5	29.37	45.0	67.25	95.5	130.0						

*Light and heavy figures indicate bending and total stresses, respectively; values below heavy line exceed 115 000 lb. per sq. in.

TABLE 37.—STRESSES IN $1\frac{5}{8}$-INCH LOCK-COIL CABLE, O X X CARRIAGE.*

(p = stress, in pounds per square inch, due to bending; E = bending modulus of elasticity for given tension; d = diameter of rope; R = radius of curvature of rope; $W = 13\frac{3}{4}$ in.; $r = 5$ in.; $p = \frac{E\,d}{2\,R}$; area = 1.779 sq. in.)

Total tension in cable, in pounds.	0.0	5 000	10 000	15 000	20 000	25 000	30 000	35 000	40 000	45 000	50 000	55 000	60 000	65 000	70 000
Tension in cable, in pounds per square inch	....	2 800	5 620	9 400	11 240	14 000	16 850	19 700	22 500	25 300	28 000	30 900	33 700	36 500	39 300
E, in millions of pounds	2.9	3.2	4.0	5.3	7.2	9.6	12.5	15.9	20.0	20.0	20.0	20.0	20.0	20.0	20.0
Angle under load O X X carriage, in degrees.	STRESS IN OUTER WIRES DUE TO BENDING, IN POUNDS PER SQUARE INCH.														
1	2 990	8 300	4 120	5 460	7 425	9 860	12 900	16 400	20 600	20 600	20 600	20 600	20 600	20 600	20 600
		6 100	**9 740**	**14 860**	**18 665**	**23 860**	**29 750**	**36 100**	**43 100**	**45 900**	**48 600**	**51 500**	**54 300**	**57 100**	**59 900**
2	5 970	6 600	8 250	10 900	14 800	19 800	25 750	34 750	41 200	41 200	41 200	41 200	41 200	41 200	41 200
		9 400	**13 870**	**20 300**	**26 040**	**33 800**	**42 600**	**54 450**	**63 700**	**66 500**	**69 200**	**72 100**	**75 900**	**77 700**	**80 500**
3	8 950	9 450	12 200	16 200	22 200	29 600	38 600	49 000	61 800	61 800	61 800	61 800	61 800	61 800	61 800
		12 250	**17 820**	**25 600**	**33 440**	**43 600**	**55 400**	**68 709**	**84 300**	**87 100**	**89 800**	**92 700**	**95 500**	**98 300**	**101 100**
4	11 930	13 200	16 500	21 800	29 600	39 500	51 200	65 500	81 500	81 500	81 500	81 500	81 500	81 500	81 500
		16 000	**22 120**	**31 200**	**40 840**	**53 500**	**68 050**	**85 200**	**104 000**	**106 800**	**109 500**	**112 400**	**115 200**	**118 000**	**120 800**
5	14 900	16 450	20 600	27 200	37 000	49 300	64 200	81 800	102 800	102 800	......	......	......	......	......
		19 250	**26 220**	**36 600**	**48 240**	**63 300**	**81 050**	**101 500**	**124 300**	**128 100**	......	......	......	......	......
6	17 850	19 700	24 600	32 600	44 800	59 100	76 900	98 000	128 200	......	......	......	......	......	......
		22 500	**30 220**	**42 000**	**55 540**	**73 100**	**93 750**	**117 700**	**145 700**	......	......	......	......	......	......
7	20 800	23 000	28 700	38 000	51 700	68 800	89 600	......	......						
		25 800	**34 320**	**47 400**	**62 940**	**82 600**	**106 450**	......	......						
8	23 800	26 300	32 800	43 500	59 100	78 800	102 700	......	......						
		29 106	**38 420**	**52 900**	**70 340**	**92 800**	**119 550**	......	......						
10	29 600	32 700	40 800	54 100	73 500	98 000	......	......	......						
		35 500	**46 420**	**63 400**	**84 740**	**112 000**	......	......	......						
12	35 500	39 200	49 000	65 000	88 250	......	......	......	......						
		42 000	**54 620**	**74 400**	**99 490**	......	......	......	......						
14	41 500	45 800	57 200	76 000	108 000	......	......	......	......						
	......	**48 600**	**62 820**	**85 400**	**114 200**	......	......	......	......						
Breaking radius, in inches, 115 000 lb. per sq. in., ultimate.	20.5	23.0	28.25	37.0	51.0	67.0	86.5	112.5	141.5						

* Light and heavy figures indicate bending and heavy stresses, respectively; values below heavy line exceed 120 000 lb. per sq. in.

tension, or *vice versa*. Below this point the cable fails by bending; above it, by the direct tension.

Fig. 21 shows the load and tension for a stress of 115 000 lb. per sq. in. in the outermost wire. The ultimate strength of the wire is 140 000 lb. per sq. in. On the left-hand edge of each curve the upper number shows the total tension and the lower number the tension in one wire.

TABLE 38.—Loads for Lock Coil Cable.

Size, in inches.	Weight per foot, in pounds.	Area, in square inches.	Tension at 30 000 lb. per sq. in.	Maximum load, in pounds.
⅞	1.8	0.50	15 000	900
1	2.5	0.70	21 000	1 250
1⅛	3.1	0.85	25 500	1 550
1¼	4.0	1.10	33 000	2 000
1⅜	4.75	1.35	40 500	2 350
1½	5.50	1.55	46 500	2 750
1⅝	6.4	1.80	54 000	3 200
1¾	7.3	2.05	61 500	3 650
1⅞	8.20	2.30	69 000	4 100
2	9.4	2.60	78 000	4 700

For cableways, and similar designs, in which the carrier does not pass the supports, moderate tensions may be used to advantage. For aerial tramways in which the cable angles over the tower saddles are small, the higher cable tensions are preferable. Considering these results in the light of experience, Table 38, of loads for lock-coil cable, is presented.

Profiles.—The calculation of aerial tramway profiles is as follows: Let Table 39 be a record of the survey of the center line, the initial station at the proposed loading terminal and topography being omitted.

TABLE 39.—Record of Survey of Center Line for Aerial Tramways.

Station.	Elevation.	Station.	Elevation.
0	6 130	15	5 660
1	6 118	deep	Gorge
2	6 077	29	5 525
3	6 058	30	5 570
4	6 062	31	5 605
5	6 015	32	5 630
6	5 948	33	5 590
7	5 920	34	5 538
8	5 898	35	5 460
9	5 868	36	5 380
10	5 855	37	5 325
11	5 835	38	5 278
12	5 807	39	5 245
13	5 752	40	5 225
14	5 715	41	5 215
15	.....	42	5 210
..	.....	43	5 200

The stations have been plotted, and are shown in Fig. 22. By using a horizontal scale of 1 in. = 500 ft. and a vertical scale of 1 in. = 200 ft., the diagram becomes of a size convenient for analysis.

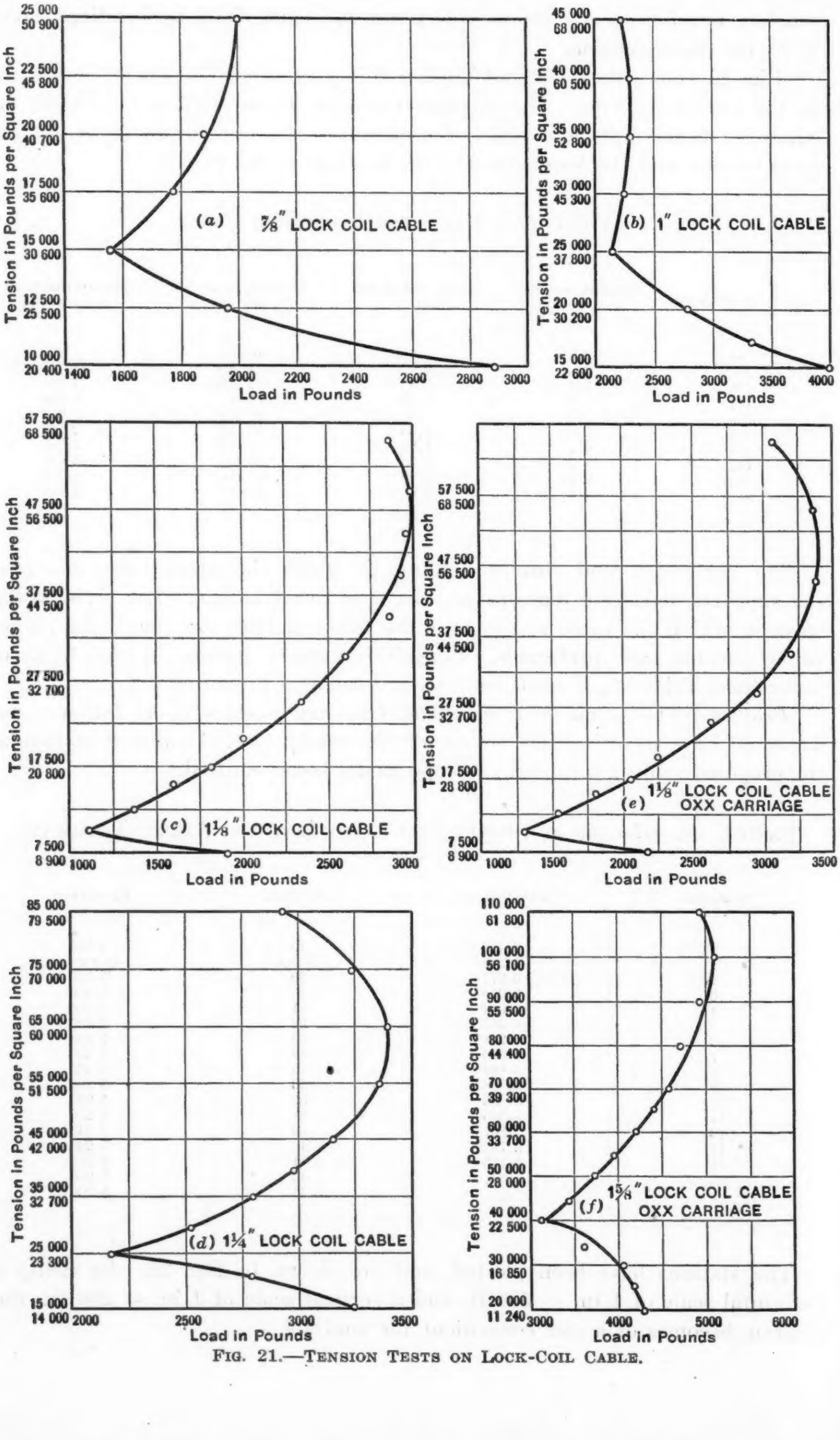

FIG. 21.—TENSION TESTS ON LOCK-COIL CABLE.

The capacity of the tramway is 72 tons per hour of ore weighing (in buckets) 100 lb. per cu. ft., with a velocity 500 ft. per min. The bucket spacing is best determined by the criterion that the cost of the carrier is equal to the cost of the track cable between carriers. Thus, if the cost of a 12-cu. ft.

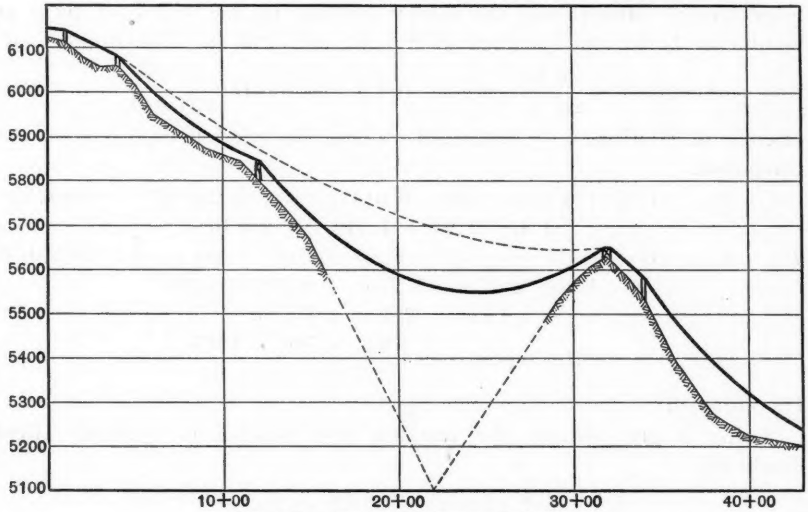

Fig. 22.—Aerial Tramway Profile.

carrier is \$150.00 and 1¼-in. locked-coil cable costs \$0.60 per ft., the spacing should be about 250 ft. As the carriers hold 1 200 lb., the spacing is calculated as follows:

$$\frac{0.03 \times 1\,200 \times 500}{72} = 250 \text{ ft.}$$

Therefore the spacing of carriers based on velocity and tonnage agrees with the cost criterion. A study of the bending stress of lock-coil cables under tension and of their frictional hysteresis, and personal experience, lead to the following conclusion: The weight of the loaded carrier plus the weight of a traction rope of a length equal to the spacing should not exceed 500 w or 525 w (w = weight per foot of cable) for track cables under tensions of 30 000 lb., or 40 000 lb. per sq. in., respectively.

The weight of an empty 12-cu. ft. carrier may be taken as 450 lb., that of ore as 1 200 lb., and 250 ft. of traction rope as 220 lb., a total of 1 870 lb. This sum, divided by 500, indicates a track cable weighing 3.8 lb. per ft., or 1¼ in. in diameter, when under 30 000 lb. per sq. in.

Assuming a snow clearance of 10 ft. for structures along the line, this profile may be divided in two sections as follows:

Section I:

Loading terminal....	Station	0	....Elev.	6 145
Curve rail	"	31 + 80	.... "	5 650
		3 180 ft.		495 ft.

Section II:

Curve rail	Station	32 + 20	....Elev.	5 645
Discharge terminal ..	"	43 + 00	.... "	5 235
		1 080 ft.		410 ft.

These data give chord slopes of 8° 51′ and 20° 46′ for Sections I and II, respectively. Multiplying the carrier spacing by the cosine of these angles results in horizontal spacings of 247 ft. and 234 ft. The number of carriers on Section I is $\frac{3\,180}{247 + 1} = 14$; and on Section II, $\frac{1\,080}{234 + 1} = 5$. The coefficient of friction, $k = 0.0141$; therefore, the traction rope tension may be computed:

$$T\,r = 1\,870\,[14\,(\sin 8^\circ\,51' - 0.0141) + 5\,(\sin 20^\circ\,46' - 0.0141)]$$
$$1\,870\,[1.95 + 1.71] = 6\,860 \text{ lb.}$$
$$T\,r - t\,r = (G - g)\,[14 \sin 8^\circ\,51' + 5 \sin 20^\circ\,46'] - (G + g) \times 19 \times 0.0141)$$
$$1\,200\,[2.155 + 1.775] - 2\,540 \times 0.268$$
$$4\,716 - 681 = 4\,035 \text{ lb.}$$
$$\text{Horse power} = \frac{4\,035 \times 500}{33\,000} = \frac{4\,035}{66} = 61$$

Therefore, 61 — 2 = 59 h. p. developed.

Using a grip sheave, the traction rope tension (to prevent slipping) should be:

$$2\,T\,r = D\,(1 + K) = 4\,035 \times 3 = 12\,105 \text{ lb.}$$

in which, $T\,r$ = 6 055 lb. This, compared with the load tension of 6 860 lb., indicates that a nominal weight for the tension mechanism may be chosen, say, 3 000 lb.

The total tension in the traction rope is then 6 860 + 1 500 = 8 360 lb. Since the stress due to bending a traction rope around a grip sheave 8 ft. in diameter is inconsequential, the size of traction rope may be tested by multiplying 8 360 lb. by a factor of safety of 4 = 33 440 lb. A 6 by 19 by ¾-in. crucible steel, Lang lay traction rope has a listed strength of 17.5 tons; therefore, the traction rope weight used in these calculations is correct.

In order to locate, approximately, the position of the important structures on the profile, the empty and loaded cable curves should be drawn. If templates are not available they may be easily prepared. Referring to the table of coefficients (Table 17) of deflections at points located at multiples of 2% of the span under 40 000 lb. tension, the corresponding coefficients for 30 000 lb. may be found by multiplying the tabular values by $\frac{4}{3}$, and a curve plotted for the empty cable, say, on a span of 5 000 ft. The loaded curve is prepared on the assumption that the weight of the carrier may be distributed along the track cable for a distance equal to the carrier spacing. In this case:

1 200 + 450 = 1 650 lb. ÷ 250 ft.	=	6.60 lb per ft.
Track cable	=	4.00 " " "
Traction rope	=	0.90 " " "
Total	=	11.50 lb. per ft.

Since a track cable with a cross-section of 1 sq. in. weighs 3.6 lb. per ft., the cable will hang as if the tension were $\frac{3.6}{11.50 \times 30\,000 \text{ lb.}}$, or 9 425 lb. In other words, the deflections are $\frac{11.50}{3.6} = 3.19$ greater than those of the empty cable. From these data the two curves are plotted and templates are prepared for the scales used on the diagram. If the diagram had been plotted with the same scale for vertical and horizontal measurements, say, 1 in. = 100 ft., the radius of circular arcs could be determined from the $\frac{t}{w}$-relationship. The radius for the empty cable is $\frac{30\,000}{3.6} = 8\,333$ ft. Divide by the scale chosen and the radius of the circular arc is 83.33 in. The loaded curve is approximated in a similar manner, namely, $\frac{30\,000}{11.50} = \frac{2\,609}{100}$ results in a radius of 26.1 in.

The curves enable the computer to locate quickly the critical points of the profile. By referring to Fig. 9, it will be noted that the empty cable has an approximate center deflection of 60 ft. and 191 ft. when loaded, or a change of more than 130 ft. If weights are to be used to maintain a constant tension, space must be provided in the station for a travel of,

$$\frac{8}{3 \times 2\,000}\,(191^2 - 60^2) = 44 \text{ ft.}$$

resulting in a lofty and expensive structure. The change in the cable slopes at the ends of the span are as follows:

$$\begin{array}{ll} \tan \beta = 0.0006 \times 2\,000, & \text{or } 6^\circ\ 50' \text{ empty cable.} \\ \quad 3.19 \times 6^\circ\ 50' & , \text{ `` } 21^\circ\ 00 \text{ loaded ``} \end{array}$$

or a difference, say, of 14 degrees.

The station hoods for transferring carriers from cable to rail cannot accommodate so great a change in slope, not to mention the many other difficulties of design and operation that will be encountered if the change in cable slopes is not limited to reasonable values. This may be done by anchoring both ends of the track cables.

When the line is in operation, the tension of the traction rope of the long span exceeds 3 200 lb., an amount sufficiently great to carry a part of the carrier weight; that is, the equivalent weight per foot of track cable is reduced from 11.50 lb. to 11.00 lb. per ft. Therefore, care must be used in stripping the line to remove alternate carriers and avoid fouling of the track cable and traction rope. Also, the track cable supporting the empty carriers must be placed under such a tension that winds will not blow it into the loaded side.

By the proper use of the templates and the deflection charts (Figs. 9 to 18), structures have been located and elevations adopted for checking by calculation. It is desirable to record these data in a simple manner and to arrange them so that the calculations follow in sequence. Such a form is shown in Table 40. For convenience the elements of design have been recapitulated.

TABLE 40.—Calculations of Profile.

(Data as follows: Capacity, 72 tons per hour; weight of material in buckets, 100 lb. per cu. ft.; velocity, 500 ft. per min.; spacing, 250 ft.; weight of loaded carrier on the line, 1 870 lb., and of empty carrier, 670 lb.; traction rope, $\frac{3}{4}$-in. crucible steel at 0.89 lb. per ft.; track cables, $1\frac{1}{4}$-in. and $\frac{7}{8}$-in. lock coil for loaded and light sides, respectively; cross-sectional areas, 1.1 and 0.5 sq. in.; at 30 000 lb. per sq. in., the tensions are 33 000 lb., and 15 000 lb. in the horizontal components of the track cables; $\frac{G}{t} = 0.0567$, or 3° 15′; the slope angle at support for empty cable is 20.7′ of arc per 100 ft.)

Type of structure.	Station.	Elevation.	tan α.	α.	Δ.	Empty cable.	Total.	Loads.	Total.	Saddle.	Radius, in feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Loading terminal	0 + 00	6 142.00									
	90	1.80	0.0200	1°09′		0°18′		3°15′			......
	0 + 90	6 140.20			3°25′		3°45′		3°15′	7° 0′	......
	10	.80	0.0799	4°34′		0° 2′		0′			
Three bent support	1 + 00	6 139.40			3°41′		3°45′		3°15′	7° 0′	......
	10	1.45	0.1450	8°15′		0° 2′		3°15′			
	1 + 100	6 137.95			2°32′		3°34′		3°31′	7°05′	......
	280	53.84	0.1905	10°47′		1° 0′		3°31′			
	3 + 90	6 084.61					— 9°47′			— 9°47′	......
Curve rail	24	7.07	0.2946	16°25′							
	4 + 14	6 077.54			17°02′		19°41′		6°54′	26°35′	70.82
	768	235.25	0.3063	17°02′		2°39′		6°54′			
	11 + 82	5 842.29			17°02′		14°23′		7°29′	7°29′	
Curve rail	36	11.00	0.3055	16°59′							......
Double anchorage	12 + 18	5 831.29					12°07′		14°10′	26°17′	114.03
	1958	184.00	0.0940	5°22′		6°45′		14°10′			
	31 + 76	5 647.29			5°22′		1°13′		14°10′	15°23′	......
Curve rail	48	2.56	0.0533	3°08′							78.65
Double anchorage	32 + 24	5 644.73			17°56′		18°00′		3°15′	21°15′	......
	76	24.60	0.3237	17°56′		0°14′		3°15′			
Tower	33 + 00	5 620.13			1°23′		1°57′		3°15′	5°12′	......
	95	33.30	0.3505	19°19′		0°20′		3°15′			
	33 + 95	5 586.83			4°32′		4°54′		3°15′	8°09′	......
Two bent structure	10	4.42	0.4421	23°51′		0° 2′		0′			
	34 + 05	5 582.41			— 2°38′		0°29′		7°40′	8°09′	......
	895	347.41	0.3881	21°13′		3°05′		7°40′			
Discharge tunnel	43 + 00	5 285.00					18°08′		7°40′	10°28′	......

Explanation of Table 40.—Column (1) indicates the type of structure, such as loading terminal, two-bent support, curve rail, and double-anchorage structure, tower, discharge terminal, etc. Column (2) indicates the survey station and the horizontal distance between. Column (3) gives the elevations of the saddles and the differences in height between them. Column (4) gives the tangents, and Column (5), the slope angles of the chords. Column (6) records the intersection angle of the chords. Plus (+) indicates downward pressure; minus (—), uplift. This angle may be changed at will by shifting the elevations of the adjoining saddles. Column (7) has the slope angle of

the tangent to the empty cable at the saddle. This angle is easily computed from the fact that cables of constant cross-section have a definite curvature per foot; for instance, a track cable weighing 3.6 lb. per ft., under a tension of 30 000 lb., curves 41.4′ of arc per 100 ft. of span, hence the slope of the tangent is 20.7′ per 100 ft. The angle of the empty cable on the saddle is the algebraic sum of the intersection and cable angles, as shown in Column (8). The former may be negative; the latter is always positive. Column (9) gives the angular increase due to the loads. These are quickly determined from the table for load tangents (Table 18). Column (10) gives the sum of the load angles at the support. Column (11) gives the result of the calculation, namely, the total angle of the loaded cable over the saddle. It is the usual aim of profile designers to impose equal cable angles on adjacent saddles. When the slope angles of cables entering or leaving structures are known, the loads imposed, and the radius of rail curvature, can be readily computed. Column (12) gives the radius of the curve tangent to the cable slopes. Experience indicates that the curvature of track cables of small diameter over saddles should be less than 9°; and less than 6° for cables 1⅜ in. or larger.

To determine the position of the empty cable on the long span, the erection tension must be computed. The simplicity of the calculation, by the method of second differences, is shown in Table 41.

Anchored Span.—The data used in computing the anchored span are as follows: Locked-coil cable, 1¼ in. in diameter, weighing 4 lb. per ft.; maximum horizontal tension, 33 000 lb.; horizontal span, 1 958 ft.; difference in terminal elevations, 184.0 ft.; weight of loaded carrier, 1 870 lb.; spacing of loads, 250 ft.; slope of chord, 5° 22′; horizontal spacing, 250 cos 5° 22′=248.91 ft.; and number of loads, 8; distance from support to first load, 107.83 ft.; and deflection of fourth load from left support:

$$y = \frac{1\,870 \times 1\,958}{4 \times 33\,000}\left[8 - \frac{64 \times 248.91}{2 \times 1\,958}\right] + \frac{4 \times 1\,958^2}{8 \times 33\,000}\left[1 - \left(\frac{248.91}{1\,958}\right)^2\right]$$

$$+ \frac{854.55}{1\,958} \times 184.0 + 80.305 = 246.51 \text{ ft.}$$

$$= 109.062 \qquad\qquad + 57.143$$

In this case the first and second differences are equal:

$$-\frac{248.91}{33\,000}\,[1\,870 + 4 \times 248.91] = 21.607 \text{ ft.}$$

$$\text{Slope factor} = \frac{248.91}{1\,958} \times 184 \qquad = \underline{23.386'}$$

$$\text{Total} \qquad = 44.993 \text{ ft.}$$

TABLE 41.—DEFLECTIONS OF ANCHORED SPAN BY FIRST AND SECOND DIFFERENCES.

Load No.	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
Deflection..........	5 831.29	5 784.58	5 696.37	5 629.77	5 584.78	5 561.39	5 559.62	5 579.44	5 620.88	5 647.29
First difference....		46.71	88.207	66.60	44.993	23.386	1.779	19.828	41.435	26.41
Second difference..		21 607	21.607	21.607	21.607	21.607	21.607	21.607	21.607	

To check Table 41, compute the deflection of the load nearest the support:

$$y = \frac{1\,870 \times 1\,958 \times 0.05507}{33\,000} [8\,(1 - 0.05507) - 0.1271 \times 28]$$

$$+ \frac{4 \times 1\,958 \times 0.05933 \times 0.05507 \times 0.94593}{2} + 0.05507 \times 184.0$$

$$= 24.460 + 12.114 + 10.133 = 46.707 \text{ ft.}$$

Therefore, the deflections given in Table 41 are correct.

The length of the loaded span is:

$$1\,958 + \frac{46.71^2 \times 26.41^2}{2 \times 107.83}$$

$$+ \frac{88.207^2 + 66.60^2 + 44.993^2 + 23.386^2 + 1.779^2 + 19.828^2 + 41.435^2}{497.82}$$

$$= 1\,958 + 13.35 + 33.95 = 2\,005.30 \text{ ft.}$$

The length of the empty span, when loaded, is increased to 2 005.30 ft., that is,

$$2\,005.30 = 1\,958 + \frac{4^2 \times 1\,958^3}{24\,t^2} + \frac{184^2}{3\,916} + \frac{0.000041 \times 1\,954}{1\,000}(33\,000 - t)$$

or,

$$t + \frac{5\,005\,000\,000}{t^2} = 36.01 + 0.00008\,t$$

Solving by slide-rule for $t = 11\,650$,

$$\frac{5\,005\,000\,000}{t^2} = 37.00$$

Similarly,

$$36.01 + 0.00008\,t = 36.95$$

Therefore, erect at a tension of 11 650 lb.

Conclusion

In conclusion, it may be stated that the foregoing remarks only hint at the magnitude and scope of aerial tramway systems of transportation. It should be emphasized that the highest degree of engineering skill and advanced mathematical analysis are required to design properly and erect a successful and economical aerial tramway. It is recalled that multi-loaded spans are not continuous functions, but a series of arcs which intersect under each carrier with an angle the tangent of which is equal to the weight of the load divided by the horizontal component of the tension. Because of this, the determinations of deflections, tangent slopes, and other properties of the cable spans have developed a system of mathematical analysis, which is as distinctive as that used in railroad location, or any other engineering specialty. In other words, aerial tramways are best designed by those who are fitted by experience and training to undertake this type of construction.

The advance in freight rates in this country makes it imperative to supply a system of transportation for industrial plants, mines, and quarries, which is safe, free from meteoric conditions, and economical, and aerial tramways meet these specifications in a most practical and satisfactory manner.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE HEAD-WORKS OF THE IMPERIAL CANAL

By C. E. Grunsky,* Past-President, Am. Soc. C. E.

Synopsis

The diversion of water from the Colorado River for the irrigation of lands in the Imperial Valley, California, has been fully described in papers previously submitted to the Society.†

In 1916, mainly on account of difficulty with the silt carried by the Imperial Canal, it became necessary to make considerable improvements which included an up-stream extension of the canal for about 1 mile and the building of an intake structure which is known as the Rockwood Gate. This gate was designed for erection on a sand foundation. It is a structure with seventy-five openings between piers and extends for more than 600 ft. along the bank of the river. For about 400 ft. of this distance, the foundation is fine sand which has been prevented from being piped out from under the gate by placing upon it, without pile support, the floor of the structure, heavily weighted by a box of sand carried by the piers which sub-divide the gate into sluice-like openings. There has been no trouble with settling during the eight years since its completion, wherefore a brief description of it now appears to be timely.

The Imperial Canal and Its Intake Structure

The Imperial Canal, so called because it was built for the irrigation of lands in that part of the Salton or Coahuila Basin now known as Imperial Valley, is also known as the Alamo Canal. Its head is in California on the right bank of the Colorado River just up stream from the International Boundary line between California and Mexico. Its course is first southerly

Note.—Written discussion on this paper will be closed in **March, 1928**.

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† "The Lower Colorado River and the Salton Basin" by C. E. Grunsky, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 1; also, "Irrigation and River Control in the Colorado River Delta," by H. T. Cory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1204.

into Mexico, thence westerly and northwesterly back into California. About 40 miles of the main canal are in Mexico. This canal supplies water to about 450 000 acres of cultivated lands in California and to about 200 000 acres in Mexico. Not an acre of the cultivated land could produce crops without irrigation, normal rainfall being only 2 to 3 in. per annum.

Within a few hundred yards of the International Boundary line, at the base of Pilot Knob which rises toward the west, is a regulating gate that was built in 1905 on good rock foundation. This was originally provided with eleven openings or sluices through which the flow was controlled by Taintor gates. Drift and silt were troublesome and interfered seriously with the manipulation of the gates, wherefore, in the course of time, several of the original gates were replaced with Stoney gates. There was an open cut, a little more than ¼ mile long, from this gate to the river, in which a dredge had to be maintained to stir up silt deposits and prevent reduction of capacity, which were always likely to occur there on a falling river. The importance of maintaining capacity will be appreciated when the fact is taken into account that about 2 000 sec-ft. of water in winter and more than 5 000 sec-ft., at the height of the irrigation season (now already in excess of 7 000 ft.) had to be sent down the canal, practically without interruption.

The water entering the canal carried from about 0.3 to 4.0%, averaging probably about 0.8%, of very fine silicious silt in suspension and, besides, had an appreciable bed or bottom load of fine sand. At the time the modification of this intake arrangement was recognized as essential, all the silt which entered the canal was carried by the water down the main canal, into the laterals, into the small irrigation ditches, and, in large part, out on the fields, lodging in the canals, ditches, and on fields, frequently at points where its accumulation became increasingly embarrassing. The quantity of this silt transported down the main canal, estimated at a weight of 100 lb. per cu. ft., was computed, from the data available, to be about 12 000 acre-ft. per year. It is now reported by M. J. Dowd, Assoc. M. Am. Soc. C. E., Chief Engineer and General Superintendent of the Imperial Irrigation District, to have reached a peak of 24 900 acre-ft. in 1923 and to have averaged more than 14 000 acre-ft. during the last 13 years.

There was a twofold purpose in the up-stream extension of the canal and the building of a gate in the bank of the river:* First, the structure was to be of such a type that it would reduce the incoming silt to a minimum, mainly by holding back the bottom load of the water; and, second, in the mile stretch of canal between the new intake gate and the original regulating gate, there would be ample space in which to operate dredges with which it was proposed to maintain extra depth, thereby providing opportunity for the settling of some of the heaviest of the silt in suspension.

The quantity of solid material in suspension in the water of the Colorado River, at Yuma, Ariz., 8 miles up stream from the head of the Imperial Canal, had been determined by the U. S. Reclamation Service which commenced its sampling in 1909. The records thus obtained established the fact that

* The project was recommended, designs were approved, and general supervision was exercised by the late George G. Anderson, M. Am. Soc. C. E., and the writer, Consulting Engineers to Imperial Irrigation District. The preliminary design was prepared by the writer.

there is somewhat less material in suspension at the water surface than at mid-depth, or near the bottom of the river. These silt determinations indicate that there is about 12% more silt in the water of the Colorado near the bottom than at the surface. It appeared, therefore, worth while to arrange the intake so that mainly surface water would enter the canal. Consequently, the gates of the Rockwood Intake were planned to be of the over-fall type, thereby not only checking the movement of silt on the bottom in the head of the canal, but also, in some measure, reducing the average quantity of silt in suspension.

Two dredges were provided for the inter-gate section of the canal and, for a time, in the period from 1918 to 1920, these removed upward of 500 000 cu. yd. per month of fine sand from this section without making any permanent impression on the depth. These facts relating to the magnitude of the silt problem are noted merely to emphasize the urgent need which had long been recognized for the Rockwood Gate. It is not proposed, however, to enter on a full discussion of the canal's silt problem at this time.

The preliminary design of the new intake structure which is built into and replaces more than 600 ft. of river bank, was made on the assumption that it would have to be constructed on a foundation of fine sand. As built, its location is at the base of a low gravel-topped hill somewhat farther up stream than had at first been planned, with the resulting advantage of securing for its up-stream one-third, or for about 200 ft., a foundation of slightly cemented sand, a material which, although quite soft, can be cut to, and will stand with, a vertical face. A similar formation, exposed at a number of places in the hills along the river at and below Yuma, is soft enough to be carved readily with a knife. Except for the large excavation required, the gate would have been shifted entirely into this cemented sand formation.

The floor of the structure is at two levels. The up-stream gates, 27 in all, have their sills 4 ft. lower than the remaining 48 down-stream gates. The gates with the lower sills are on the better foundation. The structure consists in the main of a heavy concrete floor with cut-off walls at the up-stream and down-stream edges (Fig. 1). Piers rise from the floor and are spaced so as to leave openings about 6½ ft. in the clear between their sides. The piers carry a sand-box, 22 ft. wide by 10 ft. deep, which serves the purpose of loading the floor. The weight of the fill in the sand-box, transferred by the piers to the floor, is relied on to hold in place the sand on which the structure rests.

At the up-stream edge of the structure, sheet-piling has been driven. Its purpose was not only to reduce the movement of water through the underlying sand, but also to serve as a form for the concrete cut-off walls. It will be noted that this piling gives no support to the structure, being placed in front of, and not under, the concrete so that it does not interfere with the transmission of the full weight of the intake to its sand support. It was proposed to drive this sheeting to a depth of 30 ft. below the floor level. A jet was used in driving the sheet-piles which were of the built-up Wakefield type, but so much difficulty was experienced that, in the finished work, few are more than 20 to 25 ft. in the sand. The proposition to drive them to the greater depth originated with C. R. Rockwood, Chief Engineer of the District, and

his assistants, who based their conclusion as to the possibility of doing this on long experience with similar work, although of less magnitude, in the Lower Colorado River region. In the very fine sand, at the site of the intake, the piles would drive readily about 10 or 12 ft. It was difficult to control them

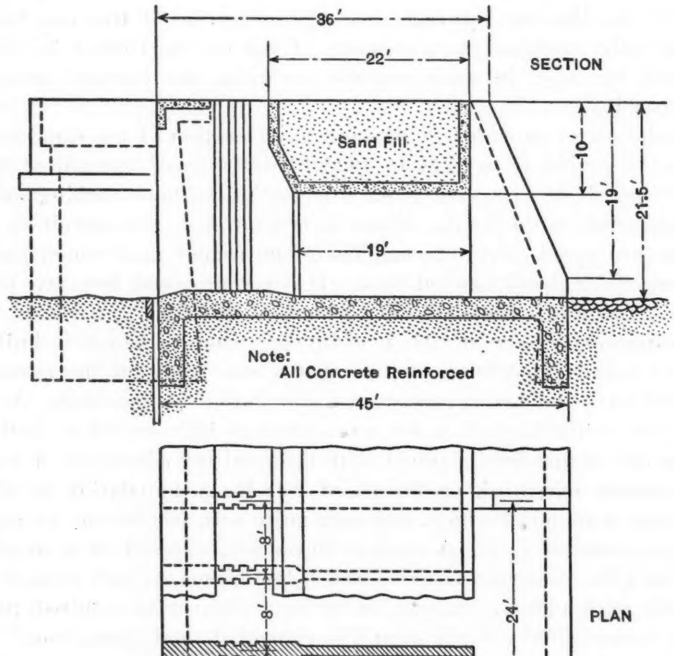

FIG. 1.—PLAN AND SECTION, ROCKWOOD GATE AT HEAD OF THE IMPERIAL CANAL, CALIFORNIA, BUILT 1918.

during further driving. They could not always be held snug against the preceding pile and despite the liberal use of water from a jet, soon required such hard blows that many of them went to pieces and had to be pulled. The sand in which this difficulty was experienced is very fine and has little, if any, admixture of clay. It appears very compact and firm under water, but partakes of the nature of quicksand when jarred or caused to vibrate.

Along the down-stream edge of the structure, at both abutments and across the canal, is a heavy rock-fill. Except for a small portion this was, according

to the original program, to be placed from time to time after the gate went into use, all settlement of the fill to be made good as it developed.

Up stream from the sand-box the piers rise several feet above the extreme high-water stage of the river and are united at the river face of the structure by a slightly ornamented cornice which gives a pleasing aspect.

Grooves are provided in the sides of the piers for flash-boards (stop-logs) over which the water drops from the river into the canal. By means of the original regulating gate in the canal, about 1 mile below the Rockwood Gate, the water in the canal is held slightly lower than the water in the river so that there may always be sufficient pressure to hold the flash-boards in place.

The quicksand foundation has given no trouble. No settling of the structure has been noted. Had the structure been supported on piles there would have been little or no load on the sand which would have yielded readily to percolating water with the probability that the gate would have been undermined. This was averted by placing the heavy weight as described directly on the sand.

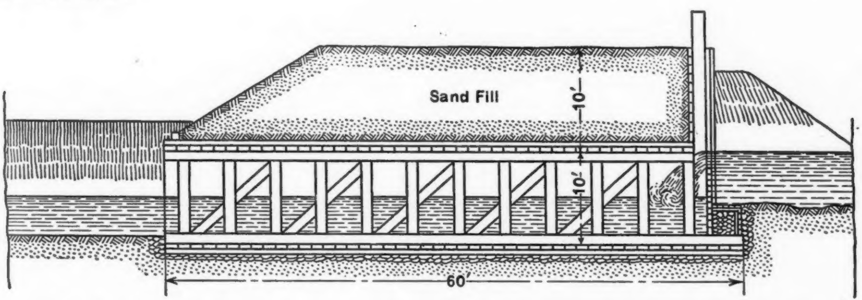

FIG. 2.—SECTION OF CHOWCHILLA CANAL REGULATOR, MERCED COUNTY, CALIFORNIA, BUILT 1877.

The prototype of this structure, Fig. 2, is a timber gate, built in the head of Chowchilla Canal, Merced County, California, in 1877. The head of this canal, on the right bank of the San Joaquin River, is in a quicksand formation. In each of four successive years timber intake structures had been placed in the head of this canal only to be undermined and destroyed by the next rise of the river. Thereupon, a ranch superintendent took the matter out of the hands of his engineers and proceeded as follows: Loose blocks of sandstone were embedded in the quicksand. On the foundation surface thus prepared, a floor of two layers of 2-in. planks was laid, the upper layer at right angles to the lower one. This floor was about 6 ft. below the bed of the river. On this floor a framework of 10 by 10-in. timber was placed which, in turn, supported a 10-ft. deep box, 10 ft. above the floor, filled with sand. Along the up-stream margin of the floor a box-like structure, 5 ft. high, filled with rock was provided, thereby creating a drop of 5 ft. from river to gate floor. The regulating gates moved between timbers which rose from the down-stream edge of this box and gave support to the head-wall of the main sand-box. The over-all length of this structure was 60 ft. The width between its side walls was 23 ft. When last seen by the writer, after it had been in service more than 20 years, it was still in good condition.

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BAFFLE-PIER EXPERIMENTS ON MODELS OF PIT RIVER DAMS

By I. C. Steele* and R. A. Monroe,† Members, Am. Soc. C. E.

Synopsis

This paper presents the results obtained from a series of experiments made on one-twentieth scale models of the diversion dams for the Pit No. 3 and Pit No. 4 hydro-electric projects of the Pacific Gas and Electric Company. The object of the experiments was to determine the most satisfactory means of controlling or destroying the energy from the over-pour water at the foot of the dams in order to prevent erosion of the down-stream banks and bed of the stream.

Several types of stilling devices were experimented with, including baffle-piers, weirs, and stilling pools of various dimensions. The most satisfactory results were obtained by the use of two rows of baffle-piers resting on a concrete apron below the bucket of the dam, the upper row of piers being truncated prisms serving as splitters and the lower row having curved up-stream faces to act as deflectors and baffles.

Pit No. 3 Dam

General Description.—Pit No. 3 diversion dam is a concrete structure of the ogee spillway type, 112 ft. high from foundation level to spill crest, with a spillway section, 18 ft. deep and 267 ft. long, designed to pass a flood of 70 000 sec-ft. The dam is arched in plan to a radius of 500 ft. Below the dam a concrete apron, with minimum thickness of 4 ft., supporting two lines

Note.—The Special Committee on Irrigation Hydraulics selected the subject of "Scouring Below Dams" as one of ten for study and research. This paper was submitted to the Committee by its authors and the Committee has recommended its publication in *Proceedings*, in order to elicit discussion of the subject (see Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1927, Society Affairs, p. 122). Written discussion on this paper will be closed in **March, 1928.**

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of baffle-piers, extends a maximum distance of 160 ft. down stream from the toe of the dam. This apron is constructed at approximately river-bed level. (See Fig. 1.)

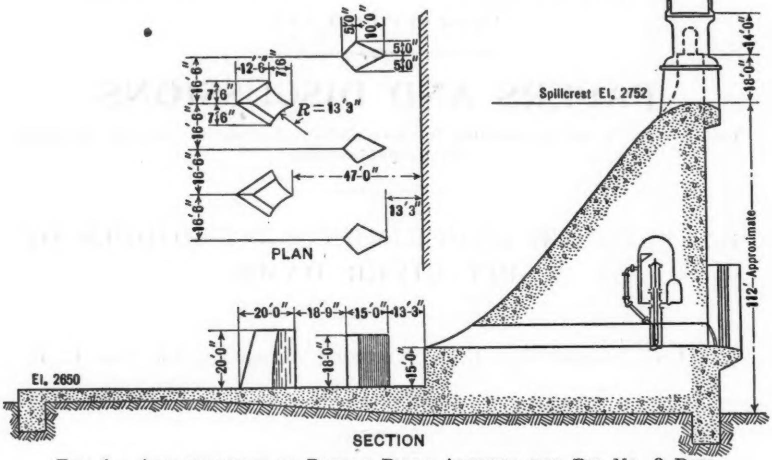


FIG. 1.—ARRANGEMENT OF BAFFLE-PIERS ADOPTED FOR PIT NO. 3 DAM.

Unusual foundation conditions at the site of the dam influenced the adoption of concrete baffle-piers in conjunction with the lining of the river channel to guard against possible scouring and erosion at the toe of the dam. The prevailing rock formations in the Pit River Basin are either basaltic or andesitic lavas, lava agglomerate, or tuff, which vary from solid rocks capable of withstanding almost any load to soft friable masses. A thorough investigation of three miles of river channel by means of diamond-drill and drift work was made in order to determine the best possible site for the dam. Explorations at the site disclosed four alternating strata of basaltic lava rock and volcanic tuff lying in nearly horizontal position, but having a slight dip down stream. It was evident that the dam would be founded for the major part on the tuff with rock abutments. Tests of small laterally unsupported cubes of the tuff showed it to have an ultimate strength equivalent to from 50 to 75 tons per sq. ft. which was considered entirely safe as the foundation pressures under the dam are only about 8 tons per sq. ft. This material is firm, dense, and impervious, and very resistant to erosion, as was evidenced by the fact that although the river had cut through the upper rock formation, it had cut into the tuff only to a small extent.

It was felt, however, that during the alternate periods of wetness and dryness some weathering would take place and also that the tremendous energy from the over-pour in times of flood might erode the tuff if left unprotected. It was not deemed advisable to attempt construction of any form of stilling pool because of the limited thickness of the tuff strata and because it was desired to leave intact as much of this impervious material as possible. Further, the banks immediately below the dam required protection, conse-

quently it was decided to line the river channel with concrete for approximately 150 ft. down stream from the dam and to construct two rows of baffle-piers placed on, and incorporated with, the apron. The toe of the dam is designed with a 15-ft. vertical face, or jump-off, which extends the full width between the concrete wing-walls. As will be noted in Fig. 1, the tangent to the curve of the down-stream face of the dam at this jump-off is downward rather than horizontal. This was so designed to reduce the base width of the dam and to direct the spill water at this point so that the stream would strike well down on the up-stream or splitter piers and downward into any high back-water. If the tangent to the curve were horizontal at this point there would be more of a tendency for the water to jump over the piers thus making them more or less ineffective at the very time they were most needed.

Tests on Model of Pit No. 3 Dam.—As there was little precedent for the use of baffle-piers, it was decided to construct a model of the dam and conduct a series of experiments to determine the proper shapes, sizes, and location of the piers. These experiments proved to be exceedingly interesting and enlightening and led to the selection of piers and layout considerably at variance with those first proposed.

Advantage was taken, in the design and construction of a small concrete diversion dam in Rock Creek, the waters of which are diverted into Pit No. 3 Tunnel, to carry out a series of experiments to determine a suitable system of baffles. This dam was designed and built as a one-twentieth scale model of the main structure in so far as features were concerned which related to the problem at hand. A simple removable flash-board arrangement was installed to store water behind the dam in order to secure a maximum head equivalent to one-twentieth of that for which the main dam was designed.

While there is some question as to the action with any given depth of flow over such a model, as compared with that for the full-scale structure and its corresponding overflow, it is believed that the results obtained clearly indicated the relative efficiency of the several layouts and that the baffle arrangement as actually constructed will give excellent results.

During the early experiments an attempt was made to measure with a Pitot tube the velocity of the water flowing over the apron. No reliable information, however, was obtained owing principally to the strong eddy currents and the foamy condition of the water. An attempt also was made to take measurements of the trajectory of the stream below the apron in order to determine by calculation the velocity of the water as it left the apron. An inclined gauge was set up for this purpose, but it was found that, due to the commotion of the water, readings could not be taken with any degree of accuracy.

It was finally decided that close observation and notation of the action of the water during the various experiments and photographic study were the best means of obtaining results. The experiments, therefore, were carried on in this manner and the notes and photographs carefully compared. Additional check experiments were then made to determine the plan that appeared to give the best results.

The experiments were made with various arrangements using three main groups of piers, as follows:

(1) Truncated pyramid-shaped piers.

(2) Prisms, or diamond-shaped piers.

(3) Combination of triangular prism piers and piers with curved up-stream faces approximating an impulse-wheel bucket in horizontal cross-section.

The spacing and types of piers, the gauge height or depth of overflow, the control height, and remarks relating to the observed results are given in Figs. 2, 3, and 4, and Tables 1, 2, and 3, for each of the eighteen experiments. Views of the action of the various piers are shown in Figs. 5 to 10 inclusive.

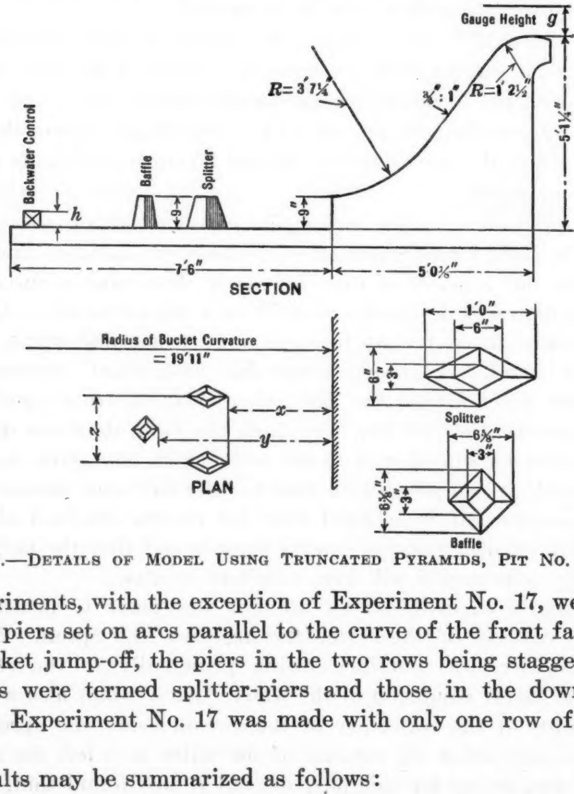


FIG. 2.—DETAILS OF MODEL USING TRUNCATED PYRAMIDS, PIT NO. 3 DAM.

All experiments, with the exception of Experiment No. 17, were made with two rows of piers set on arcs parallel to the curve of the front face of the dam and the bucket jump-off, the piers in the two rows being staggered. The up-stream piers were termed splitter-piers and those in the down-stream row, baffle-piers. Experiment No. 17 was made with only one row of curved baffle-piers.

The results may be summarized as follows:

1.—The first group of experiments using truncated pyramids gave very unsatisfactory results, the water leaping very high into the air as it left the splitter-piers.

2.—The second group of experiments using prism piers gave better results but were still unsatisfactory.

3.—The third group of experiments using curved baffle-piers showed a marked improvement in stilling action. The best results were obtained by locating the splitter piers close to the jump-off. Experiment No. 11, made to

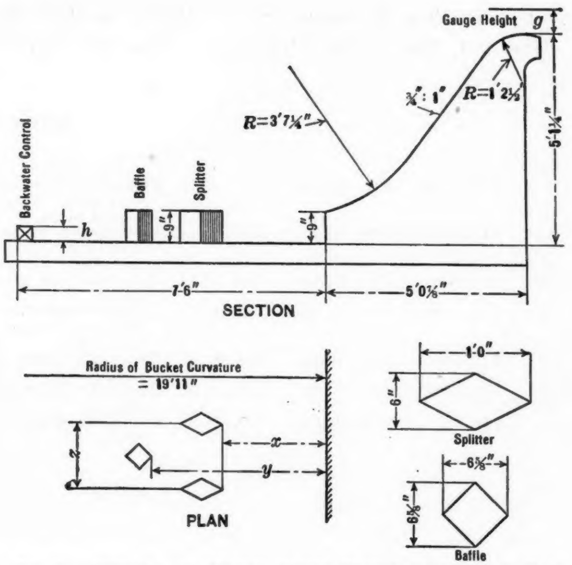


FIG. 3.—DETAILS OF MODEL USING PRISMS, PIT NO. 3 DAM.

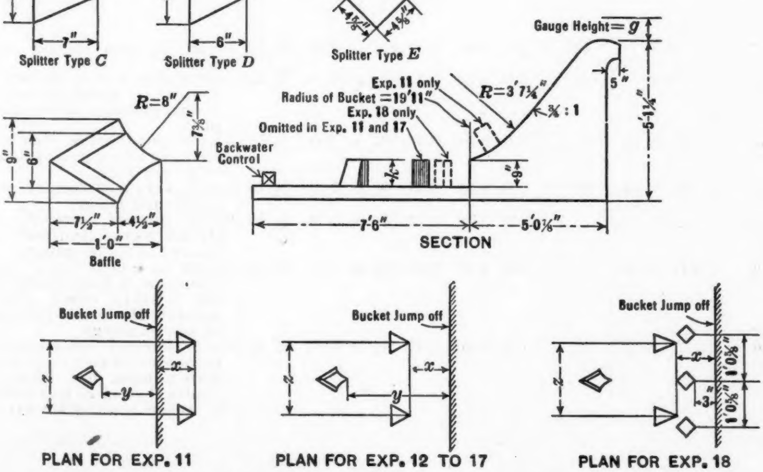


FIG. 4.—DETAILS OF MODELS USING CURVED BAFFLE-PIERS, PIT NO. 3 DAM.

TABLE 1.—BAFFLE-PIER EXPERIMENTS ON MODEL OF PIT NO. 3 DAM. GROUP 1, TRUNCATED PYRAMIDS. (SEE FIG. 2.)

Experiment No.	*g.*	*h.*	*x.*	*y.*	*z.*	Stilling action.
1	9 in	0 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Velocity of water leaving apron very high. Water hurdled second row of piers.
2	9 in.	4 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Velocity of water leaving apron very high. Water hurdled second row of piers.
2	7 in.	4 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Action not as good as with 9-in. gauge.
2	6 in.	4 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Action better than at 7-in. gauge. Hydraulic jump started at 3⅛-in. gauge.
3	....	....	3 ft. 0 in.	4 ft. 2¾ in.	7 ft. ¼ in.	Hydraulic jump started at 3-in. gauge.
4	....	....	1 ft. 6 in.	2 ft. 8¾ in.	7 ft. ⅞ in.	Hydraulic jump started at 2¾-in. gauge.
5	....	....	2 ft. 6 in.	3 ft. 8¾ in.	9 ft. ⅝ in.	Hydraulic jump started at 3¼-in. gauge.

TABLE 2.—BAFFLE-PIER EXPERIMENTS ON MODEL OF PIT NO. 3 DAM. GROUP 2, PRISMS. (SEE FIG. 3.)

Experiment No.	*g.*	*h.*	*x.*	*y.*	*z.*	Stilling action.
6	9 in.	4 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Water hurdled second row of piers.
6	6 in.	4 in.	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Action improved as gauge height decreased. Comb of hydraulic jump appeared up stream from piers.
7	5 ft. ¾ in.	0	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Action good below this gauge. Velocity of water leaving apron still very high.
7	3 ft. ¾ in.	0	2 ft. 6 in.	3 ft. 8¾ in.	7 ft. ½ in.	Action very satisfactory at this gauge and below.
8	....	....	2 ft. 6 in.	3 ft. 8¾ in.	9 ft. ⅝ in.	Water jumped vertically at up-stream piers. Hydraulic jump started at 3¼-in. gauge.
9	4 ft. ½ in.	....	2 ft. 6 in.	3 ft. 8¾ in.	1 ft. 4¾ in.	Height to which water jumped less than in Experiment No. 8, but velocity below piers was greater. Hydraulic jump started at 3-in. gauge.
10	....	....	1 ft. 0 in.	3 ft. 8¾ in.	9 ft. ⅝ in.	Water hurdled from up-stream piers over down-stream piers at high gauges, but did not rise higher than in Experiment No. 8. Action unsatisfactory.

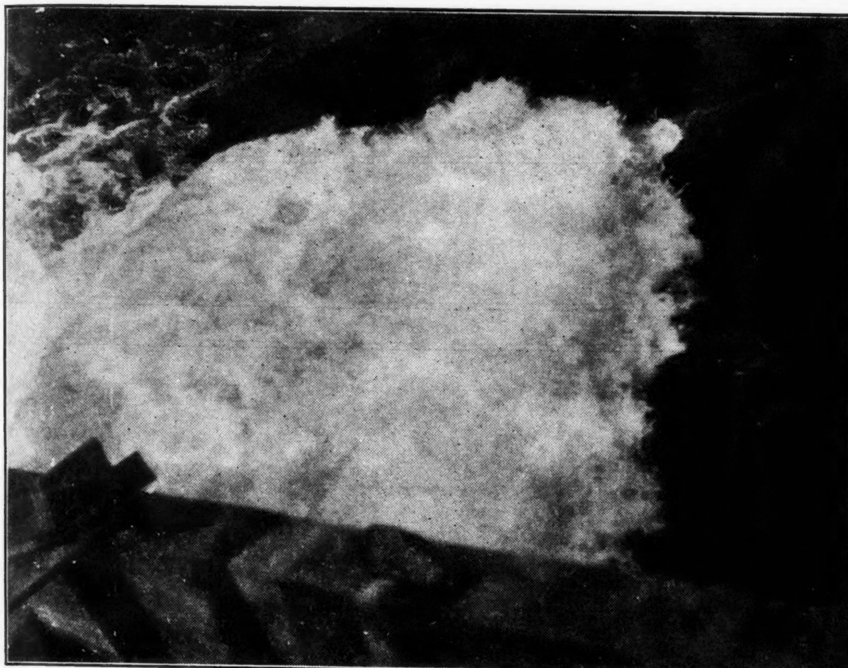


Fig. 5.—Pit No. 3 Test Dam: Experiment No. 1. Truncated Pyramid Piers. No Back-Water Control. Gauge Height Proportional to 15 Feet on Actual Dam.

Fig. 6.—Pit No. 3 Test Dam: Experiment No. 2. Same as Fig. 5, with Back-Water Control Proportional to 6 Feet 8 Inches on Actual Dam.

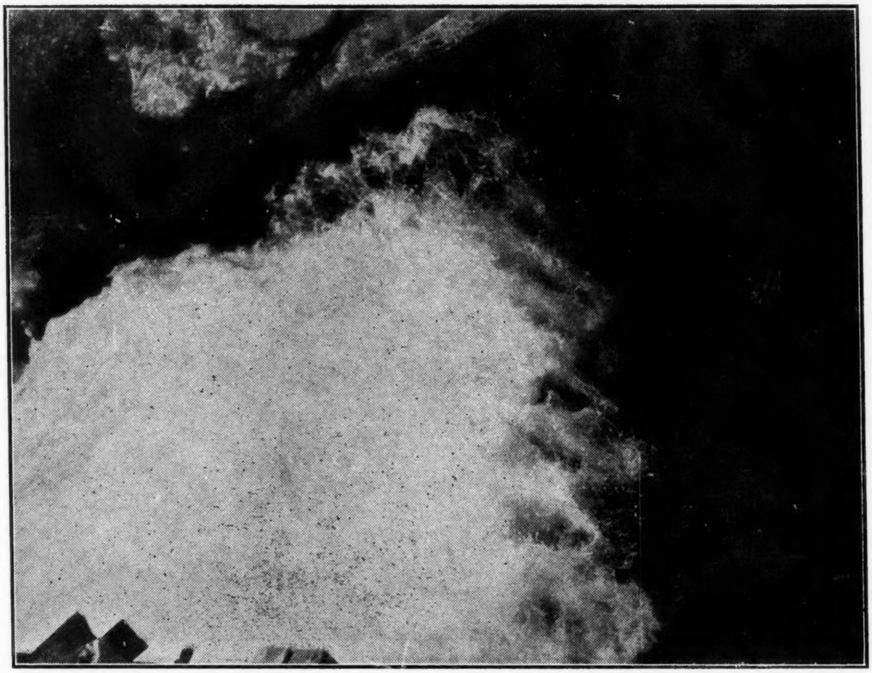


FIG. 7.—PIT NO. 3 TEST DAM: EXPERIMENT NO. 6. PRISMATIC PIERS. BACK-WATER CONTROL PROPORTIONAL TO 6 FEET 8 INCHES AND GAUGE HEIGHT TO 15 FEET ON ACTUAL DAM.

FIG. 8.—PIT NO. 3 TEST DAM: EXPERIMENT NO. 6. SAME AS FIG. 7, BUT WITH GAUGE HEIGHT PROPORTIONAL TO 10 FEET ON ACTUAL DAM.

P

FIG

FIG.

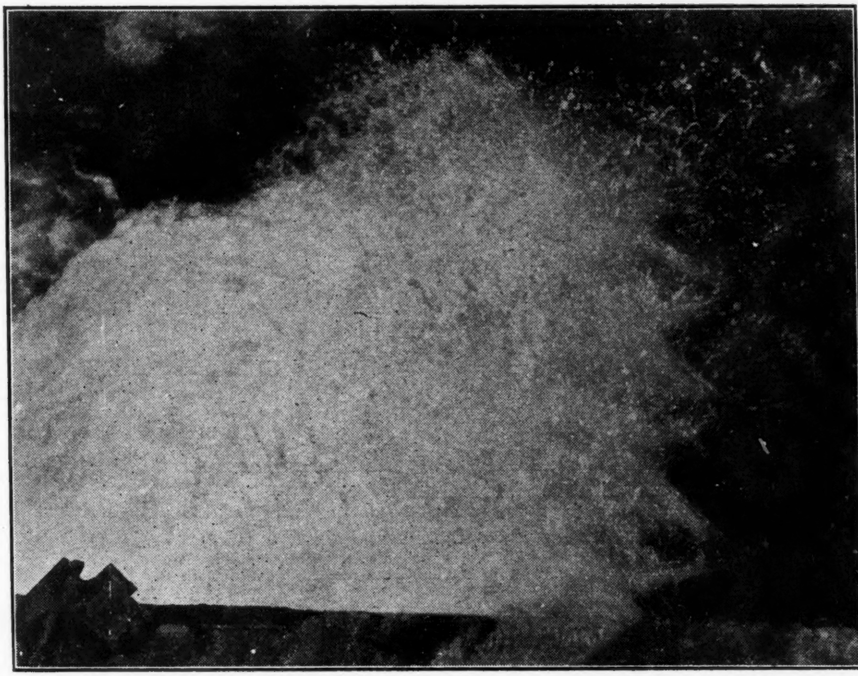


FIG. 9.—PIT NO. 3 TEST DAM: EXPERIMENT NO. 11. SPLITTER PIERS ON TOE OF DAM. CURVED BAFFLE-PIERS ON APRON. BACK-WATER CONTROL PROPORTIONAL TO 6 FEET 8 INCHES AND GAUGE HEIGHT TO 15 FEET ON ACTUAL DAM.

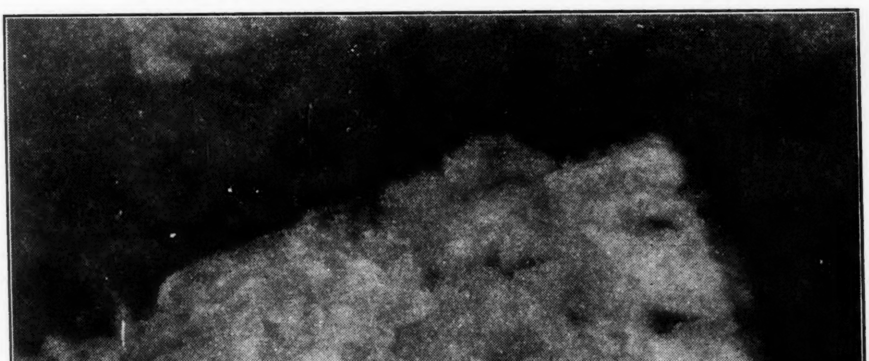
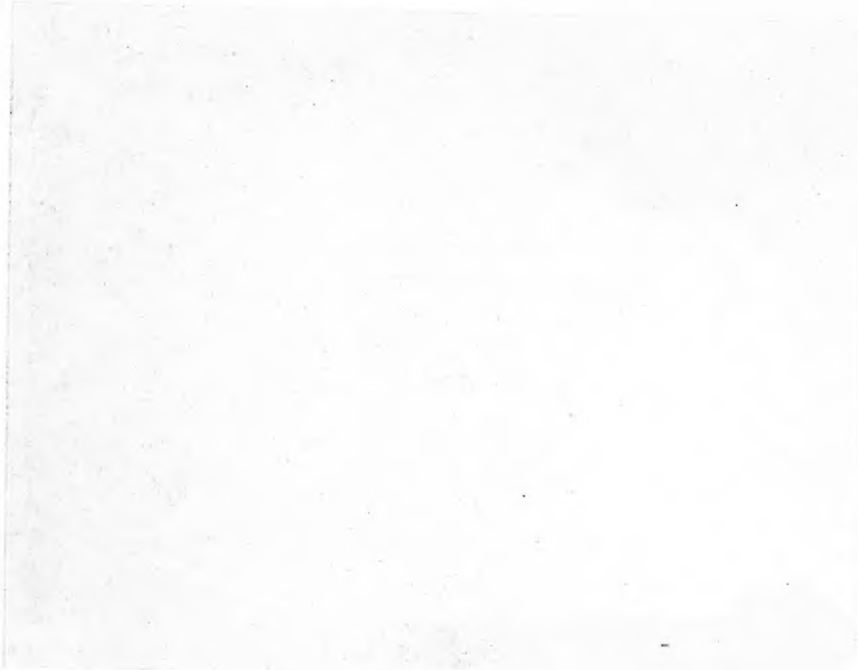


FIG. 10.—PIT NO. 3 TEST DAM: EXPERIMENT NO. 13. SPLITTER PIERS AND CURVED BAFFLE-PIERS ON APRON. BACK-WATER CONTROL PROPORTIONAL TO 6 FEET 8 INCHES AND GAUGE HEIGHT TO 10 FEET ON ACTUAL DAM.



determine the effect of placing the splitter piers on top of the jump-off, gave poor results. The water striking against these piers splashed very high on either side and in a manner very similar to the sidewash at the bow of a fast moving boat.

TABLE 3.—BAFFLE-PIER EXPERIMENTS ON MODEL OF PIT NO. 3 DAM. GROUP 3, CURVED BAFFLE-PIERS. (SEE FIG. 4.)

Experiment No.	*g*.	*h*.	*x*.	*y*.	*z*.	Type of splitter.	*k*.	Stilling action.
11	9 in.	4 in.	7 in.	1 ft. 0 in.	1 ft. 9 in.	D	9 in.	Results very unsatisfactory. Water splashed very high at splitter-piers.
11	6 in.	4 in.	7 in.	1 ft. 0 in.	1 ft. 9 in.	D	9 in.	Results very unsatisfactory.
12	9 in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	D	9 in.	Action better, but still unsatisfactory.
12	7 in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	D	9 in.	Action still better, but unsatisfactory.
12	5 in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	D	9 in.	Very good for gauges below this point.
13	9 in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	C	9 in.	Action excellent.
13	6 in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	C	9 in.	Action excellent.
13	4½ in.	4 in.	8 in.	2 ft. 4 in.	1 ft. 9 in.	C	9 in.	Action excellent.
14	......	....	1 ft. 6 in.	2 ft. 6 in.	1 ft. 0½ in.	C	9 in.	At high gauges, water washed over tops of front piers and hurdled curved piers. Action good at lower stages.
15	......	....	1 ft. 6 in.	2 ft. 6 in.	1 ft. 0½ in.	C	1 ft. 0 in.	Did not wash over piers, but jumped 3 ft. 9 in. vertically from them at gauge heights of 13 ft. 0 in., or more. Very good hydraulic jump occurred up stream from piers and between rows below 5⅝-in. gauges.
16	......	....	1 ft. 6 in.	4 ft. 6 in.	1 ft. 1 in.	C	1 ft. 0 in.	Good. Curved piers caused good stilling action.
17	9 in.	4 in.	2 ft. 4 in.	....	1 ft. 9 in.	....	....	Action good. Did not compare with Experiment No. 13.
18	......	....	1 ft. 0 in.	2 ft. 0 in.	1 ft. 9 in.	C and E	1 ft. 0 in.	Additional piers 3 in. from jump-off. Action good. Splashed to 2 ft. 3 in. height.

The layout finally adopted in the design and construction of the Pit No. 3 diversion dam gave excellent results. This was approximately that used in Experiment No. 13. The water flowing over the lower edge of the apron was comparatively quiet for all gauge heights and from all appearances no more harmful than the natural stream would be in time of high water.

Behavior of Completed Structure.—The Pit No. 3 Dam was completed in the summer of 1925 and, consequently, has passed through only two seasons of high flow. No floods were experienced in the spring of 1926 or 1927, the maximum flow of the river amounting to less than 8 000 sec-ft. The stilling action of the piers proved to be excellent. Continued observation during periods of high water will be maintained and eventually this structure will afford valuable evidence of the success or failure of this method of preventing scour below dams.

PIT NO. 4 DIVERSION DAM

General Description.—Pit No. 4 diversion dam is situated on Pit River approximately 9 miles down stream from Pit No. 3 diversion dam. On

account of the peculiar foundation conditions at the site, the dam is designed as a composite structure. The foundation on the right bank and under the main channel of the stream is solid rock, whereas the left bank consists of a compact soil, gravel, and boulder formation. The design adopted for this site consisted of a concrete ogee overflow spillway section from the right bank across the main stream channel, flanked by a flat-slab and buttress non-overflow structure on the left bank.

This dam is a relatively low structure, its function being to raise the water surface of the stream about 40 ft. The spill section of the dam is 136 ft. long, controlled by two drum-gates, each 68 ft. long by 14½ ft. high. Provision is made in the design for a maximum overflow depth of 27½ ft., which it is estimated will discharge a flood of 70 000 sec-ft.

Below the dam a two-level concrete apron supporting two lines of baffle-piers extends a distance of 100 ft. down stream. The apron at the lower level has incorporated in its design a stilling pool, but this was incidental and was adopted as a measure of economy on account of the slope of the rock foundation and also to accommodate more conveniently the discharge openings through the dam. Fig. 11 shows a cross-section of the dam through the maximum or stilling-pool section and also indicates in plan the arrangement adopted for the baffle-piers.

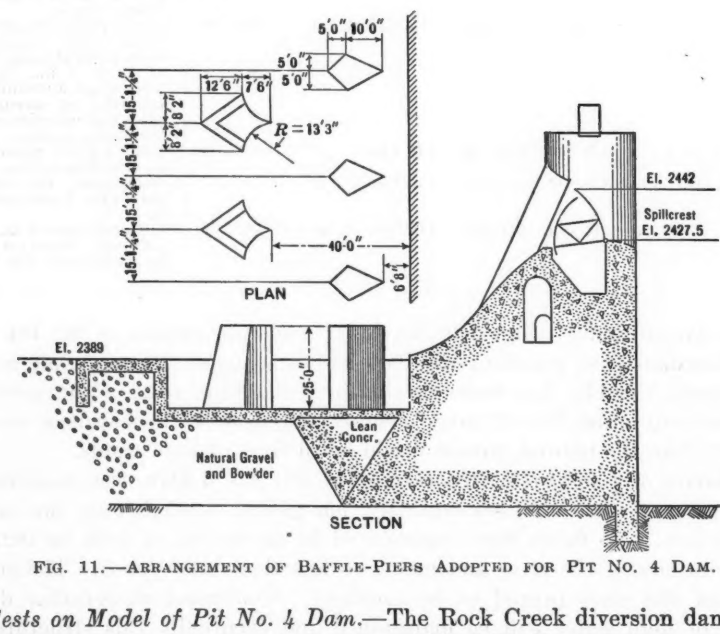


FIG. 11.—ARRANGEMENT OF BAFFLE-PIERS ADOPTED FOR PIT NO. 4 DAM.

Tests on Model of Pit No. 4 Dam.—The Rock Creek diversion dam which was constructed as a one-twentieth scale model of Pit No. 3 Dam, was reshaped with a wooden overflow and apron section to correspond to a one-twentieth scale model of the proposed Pit No. 4 Dam. A length of channel representing 100 ft. of river channel down stream from Pit No. 4 was also constructed to the shape of the actual river with loose rocks on a false wooden bottom. The general arrangement of the test model is shown by Fig. 19.

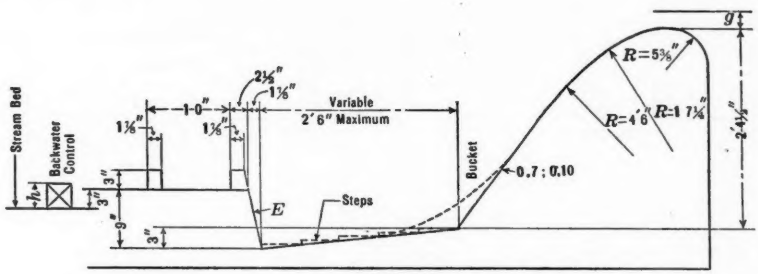


FIG. 12.—DETAILS OF MODELS WITH STILLING POOL, PIT NO. 4 DAM.

Various test runs were made by placing flash-boards across the overflow section of the dam and allowing the water in the reservoir above to rise to a gauge height corresponding to 28 ft. on the proposed Pit No. 4 Dam. This

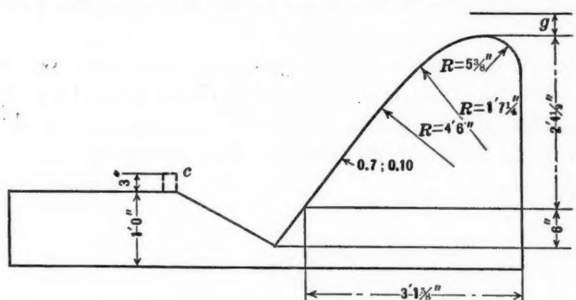


FIG. 13.—DETAILS OF MODEL WITH FRENCH DESIGN SETTLING POOL, PIT NO. 4 DAM.

allowed the overflow water, when the flash-boards were lifted, to come to a steady condition of flow at 27½ ft. representing the maximum expected flood.

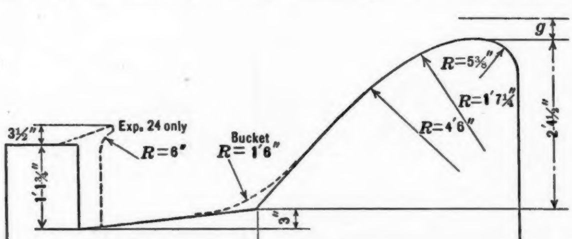


FIG. 14.—DETAILS OF MODELS FOR MISCELLANEOUS TESTS, PIT NO. 4 DAM.

Experiments were made with the five main groups, or layouts, as follows:

Group No. 1.—Stilling-pool design (Fig. 12).
Group No. 2.—The French stilling pool with upward sloping bottom (Fig. 13).
Group No. 3.—Miscellaneous layouts (Fig. 14).
Group No. 4.—Baffle-pier designs (Figs. 15 and 16).
Group No. 5.—Baffle-pier designs, with two-level apron (Fig. 17).

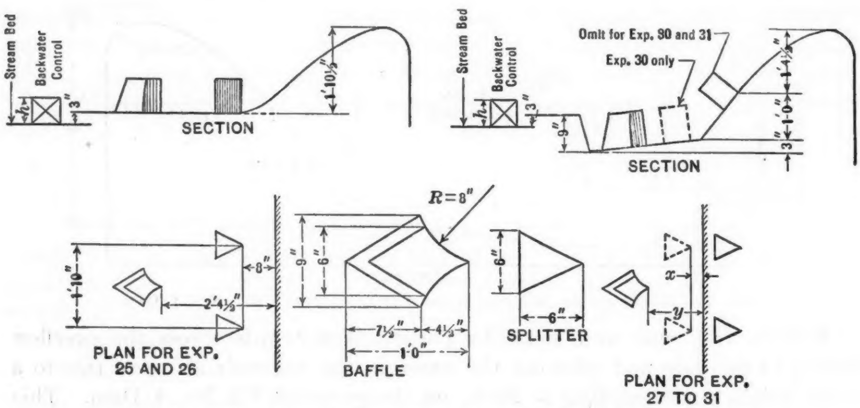


FIG. 15.—DETAILS OF MODELS WITH BAFFLE-PIERS, PIT NO. 4 DAM.

Under each of these main layouts various tests were run as given in Tables 4 to 9, inclusive, and indicated by the views, Figs. 18 to 27, inclusive, numbered to correspond with the respective experiments as shown on the photographs. In all, a total of forty-four test runs was made in connection with the design of this dam.

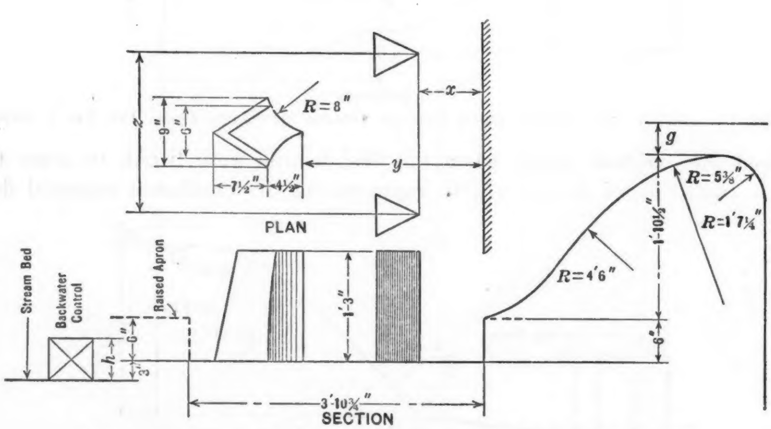


FIG. 16.—DETAILS OF MODEL WITH BAFFLE-PIERS, PIT NO. 4 DAM.

The site was favorably situated to permit the construction of a stilling pool and this type of design was first tested. However, the stilling action observed did not compare favorably with that which had been observed for baffle-piers of the Pit No. 3 tests, and after experimenting with the plain stilling pool, the French stilling pool, and some miscellaneous layouts, it was decided to experiment with baffle-piers. The piers were duplicates of those at Pit No. 3 and spaced similarly. Splitter piers, that is, the up-stream piers, were half spaced from the wing-walls, and the end pier in the line of curved piers was one-half the regular pier fastened against the wing-wall on one side.

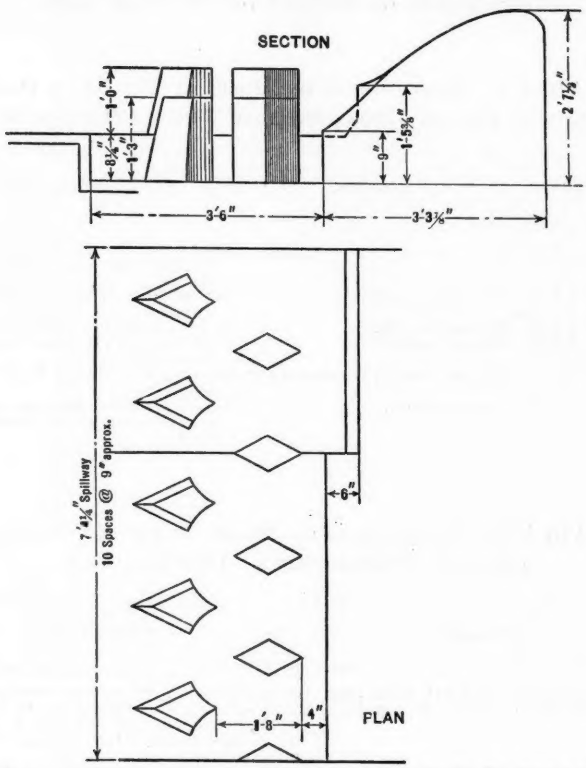


FIG. 17.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM, GROUP 5, TWO-LEVEL APRONS.

TABLE 4.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM. GROUP 1, STILLING POOL. (SEE FIG. 12.)

Experiment No.	*h*.	Length of pool.	Remarks.	Stilling action.
1	....	2 ft. 6 in.	With bucket, without weirs.	Action poor.
2	....	2 ft. 6 in.	With bucket, without weirs, *g* = 11⅜ in.	Action poor.
3	....	2 ft. 6 in.	With bucket and with both weirs.	Action poor. Water rose higher than without weirs.
4	....	2 ft. 6 in.	Without bucket and with both weirs.	Action practically same as Experiment No. 3.
5	....	2 ft. 6 in.	With steps, without bucket, and without weirs.	Steps had no effect.
6	....	2 ft. 6 in.	With steps, without bucket, and with both weirs.	Weirs only served to raise water higher.
7	....	2 ft. 3 in.	Without bucket, without weirs.	Shortening pool had no effect.
8	....	2 ft. 0 in.	Without bucket, without weirs.	Shortening pool had no effect.
9	....	2 ft. 0 in.	Without bucket, without weirs. Face *K* vertical.	Action poor.
10	....	2 ft. 0 in.	Without bucket. Vertical face of weir at vertical face, *E*.	Water rose even higher than without weir.
11	11⅜ in.	2 ft. 0 in.	Without bucket, without weirs.	Action poor. Water rose to 2 ft. 0 in. Control did not back water up to pool on account of high velocity.
12	11⅜ in.	2 ft. 0 in.	Without bucket and with both weirs.	Action same as Experiment No. 11, except water rose to 2 ft. 3 in.
13	....	9 in.	Without bucket, without weirs.	Water washed over pool until gauge was down to 1 ft. 0 in. when hydraulic jump started.

TABLE 5.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.
GROUP 2, FRENCH DESIGN STILLING POOL. (SEE FIG. 13.)

Experiment No.	Slope.	Remarks.	Stilling action.
14	2 : 1	....	Action very poor. Water jetted smoothly through pool and jumped into river channel.
15	1.5 : 1	....	Action better than Experiment No. 14, but, still poor.
16	1.5 : 1	Weir at Face *C*.	Water rose higher than Experiment No. 15.
17	1.5 : 1	11⅝-in. control.	Water rose to 1 ft. 9 in. High velocity below apron.
18	1.5 : 1	Weir at Face *C*; 11⅝-in. control.	Same as Experiment No. 17, except water rose higher.
19	1.5 : 1	5¼-in. control.	Action unstable, showing this control not as good as higher one used in Experiment No. 17.

TABLE 6.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.
GROUP 3, MISCELLANEOUS. (SEE FIG. 14.)

Experiment No.	Remarks.	Stilling action.
20	Without bucket, pool, 2 ft. 4¾ in. long.	Action very poor. Water rose very high.
21	Without bucket, pool, 1 ft. 3 in. long.	About same as Experiment No. 20, but less stable. At times water jetted over pool as in Experiment No. 13 until down to 1 ft. 0 in. gauge when hydraulic jump started to act.
22	With bucket, pool, 2 ft. 7¼ in. long.	Action only slightly better than in Experiment No. 23 below.
23	With bucket, pool, 2 ft. 0 in. long.	Action not good. Very stable rising of water at up-stream edge of raised apron.
24	Without bucket, pool, 2 ft. 0 in. long. Curved up-stream face of apron.	Action very bad. Water rose 2 ft. 3 in. to 2 ft. 6 in. and washed down-stream of apron.

TABLE 7.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.
GROUP 4, BAFFLE-PIERS. (SEE FIG. 15.)

Experiment No.	*h*.	*x*.	*y*.	Remarks.	Stilling action.
25	......	8 in.	2 ft. 4¼ in.	....	Action poor. Water splashed to 2 ft. 6 in. height and down into river channel.
26	6⅞ in	8 in.	2 ft. 4¼ in.	....	Action same as in Experiment No. 25.
27	11⅝ in.	....	9 in.	Splitter on dam.	Action fairly good. Water rose 1 ft. 6 in. to 1 ft. 8 in. Slightly greater velocity in river channel than with Experiment No. 32 to 39.
28	5¼ in.	....	9 in.	Splitter on dam.	Action practically same as in Experiment No. 27.
29	11⅝ in.	....	1 ft. 1¼ in.	Splitter on dam.	Action not as good as in Experiment No. 27.
30	11⅝ in.	3 in.	1 ft. 7¼ in.	Splitter in pool.	Action poor. Water rose to 2 ft. 0 in.
31	11⅝ in.	....	1 ft. 7¼ in.	No splitter.	Poor stilling action. Water rose to 2 ft. 0 in. and had considerable velocity in river channel.

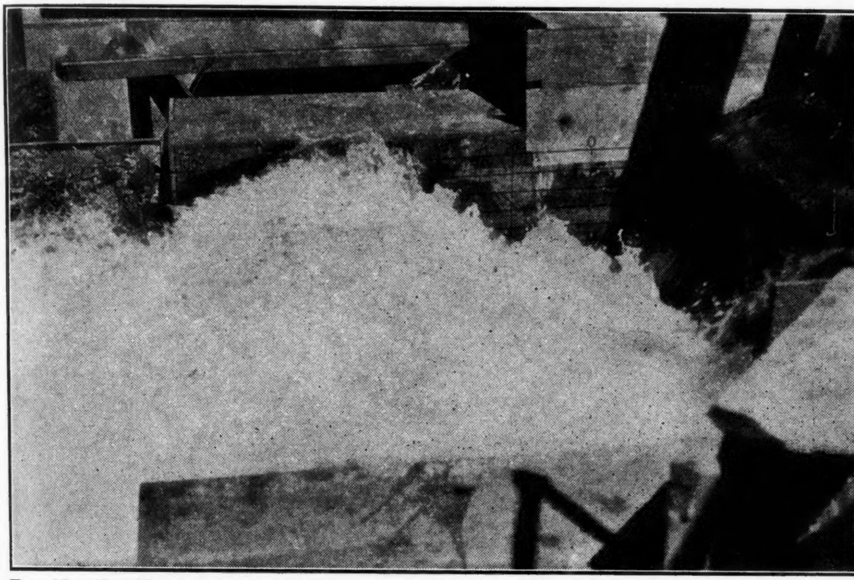


FIG. 18.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 1. STILLING POOL WITH BUCKET. GAUGE HEIGHT PROPORTIONAL TO 27 FEET 6 INCHES ON ACTUAL DAM.

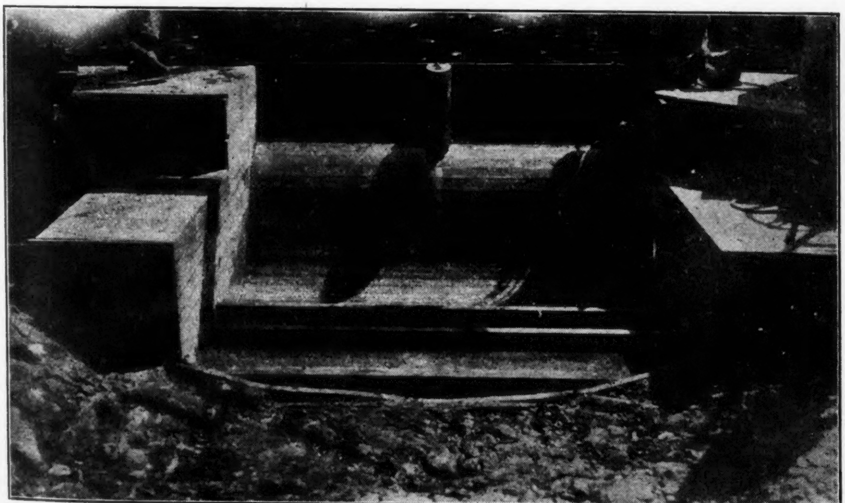


FIG. 19.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 3. SET-UP FOR STILLING POOL WITH BUCKET AND TWO WEIRS.

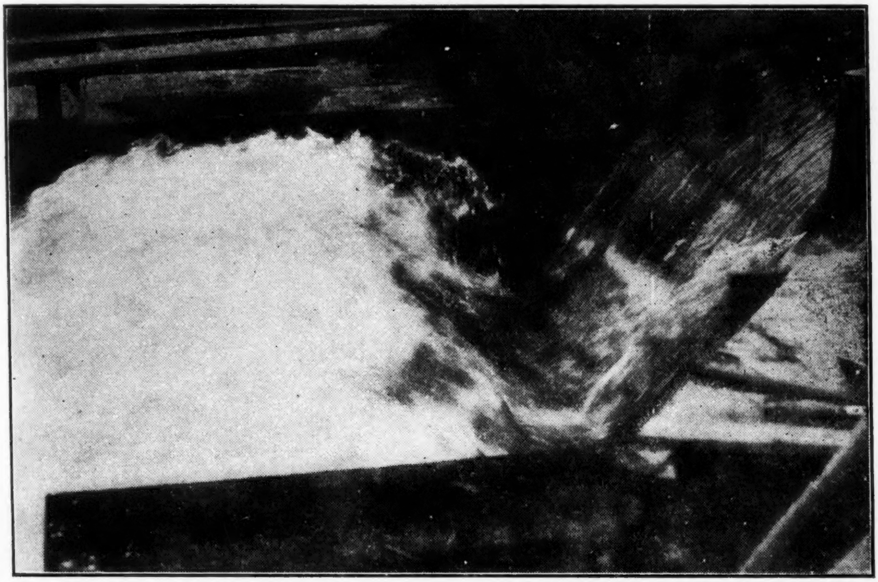


FIG. 20.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 15. FRENCH DESIGN. 1.5:1 POOL SLOPE.

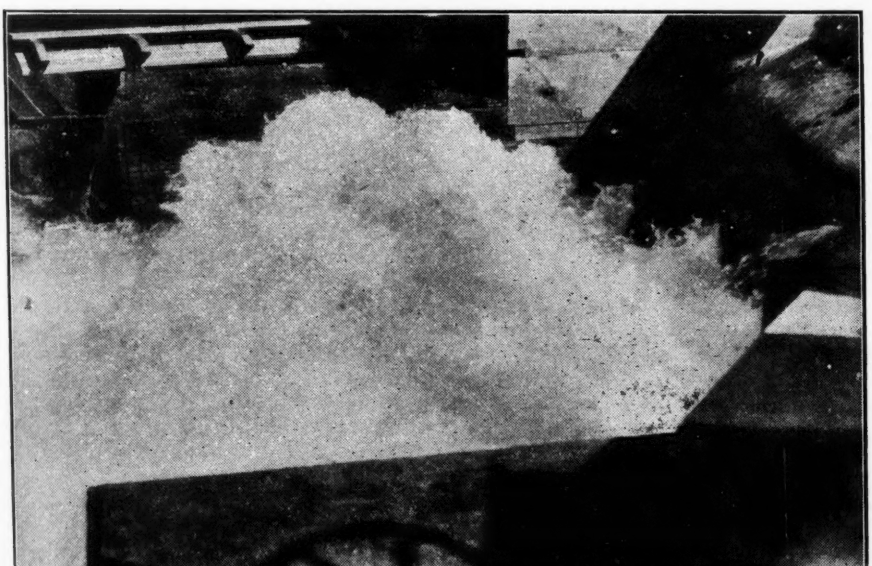


FIG. 21.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 16. SAME AS FIG. 20, WITH ONE WEIR AT TOP OF SLOPE.

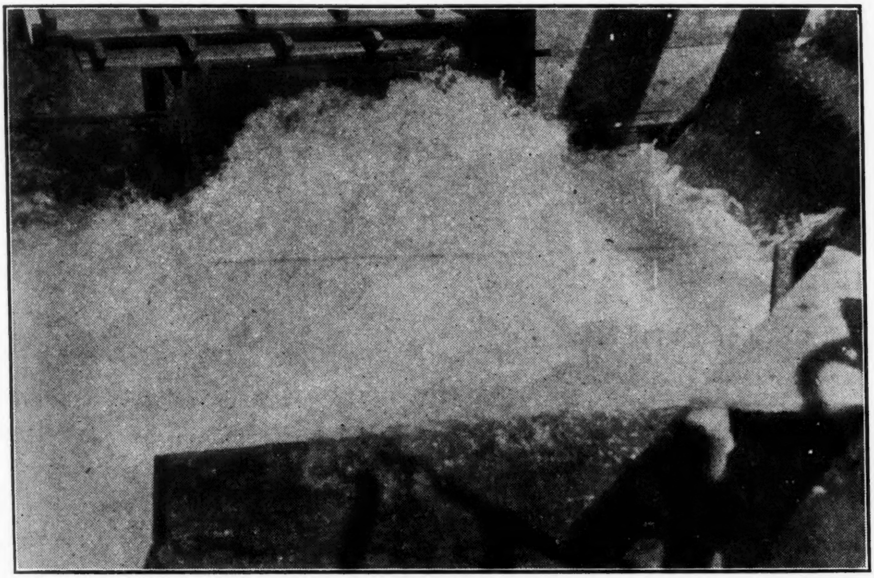


FIG. 22.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 20. MISCELLANEOUS TYPES. NO BUCKET ON DAM. VERTICAL FACE AT APRON.

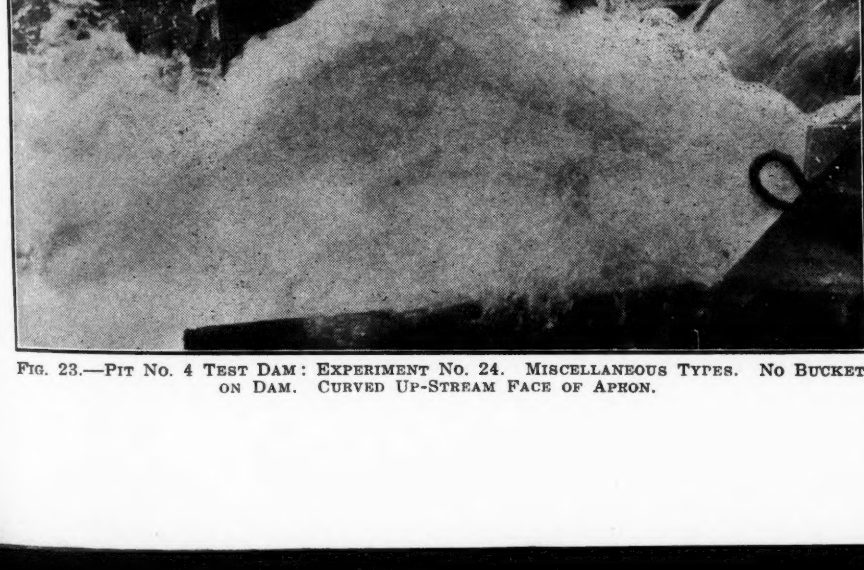


FIG. 23.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 24. MISCELLANEOUS TYPES. NO BUCKET ON DAM. CURVED UP-STREAM FACE OF APRON.

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FIG. 24

SPLI

FIG. 25.—

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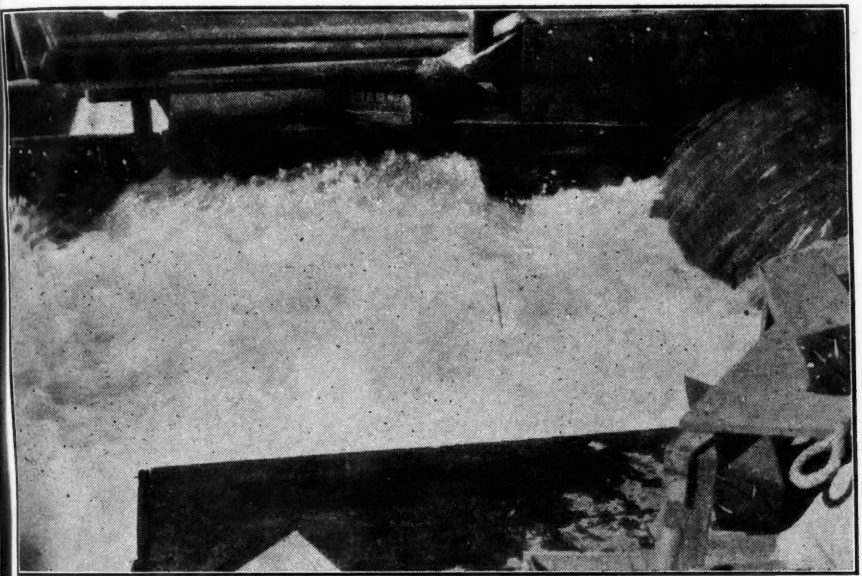

FIG. 24.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 31. CURVED BAFFLE-PIERS WITHOUT SPLITTER PIERS. BACK-WATER CONTROL PROPORTIONAL TO 19 FEET ON ACTUAL DAM.

FIG. 25.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 32. CURVED BAFFLE-PIERS WITH SPLITTER PIERS AND CURVED BAFFLE-PIERS. BACK-WATER CONTROL PROPORTIONAL TO 11 FEET 6 INCHES ON ACTUAL DAM.

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FIG

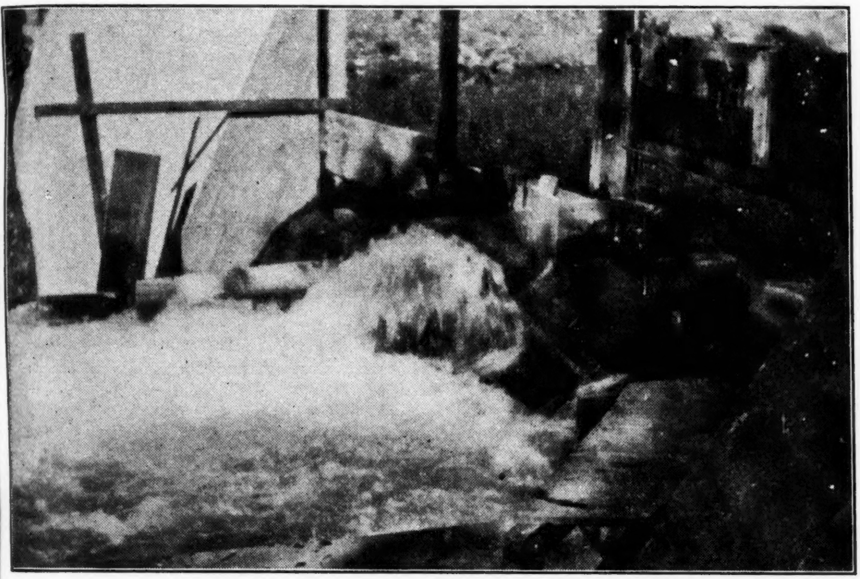

FIG. 26.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 38. TWO-LEVEL APRONS WITH BAFFLE-PIERS.

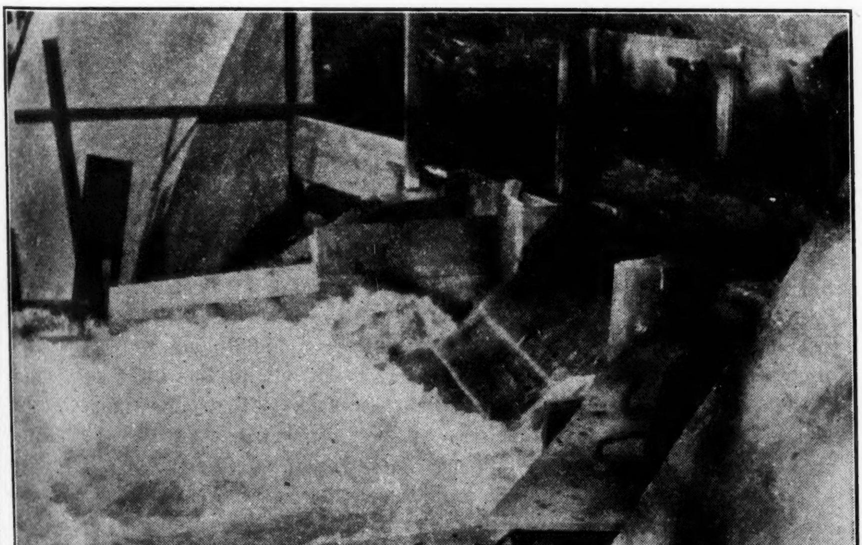

FIG. 27.—PIT NO. 4 TEST DAM: EXPERIMENT NO. 40. SAME AS FIG. 26, EXCEPT RIGHT SPLITTER PIER REMOVED.

TABLE 8.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.
GROUP 4, BAFFLE-PIERS (Continued). (SEE FIG. 16.)

Experiment No.	h.	x.	y.	Remarks.	Stilling action.
32	6⅞ in.	8 in.	2 ft. 4¼ in.	....	Stilling action very good.
33	6⅞ in.	8 in.	2 ft. 4¼ in.	$g = 1$ ft. 0 in.	Stilling action very good.
34	......	4 in.	2 ft. 0¼ in.	River channel extended upstream to within 2 in. of baffle-piers.	Action practically same as Experiment No. 32.
35	9 in.	4 in.	2 ft. 0¼ in.	River channel extended upstream to within 2 in. of baffle-piers.	Action very good. Water below apron quiet.
36	5¼ in.	4 in.	2 ft. 0¼ in.	Raised apron.	Action very good. Water rose from 1 ft. 3 in. to 1 ft. 6 in.
37	11⅜ in.	4 in.	2 ft. 0¼ in.	Raised apron.	Action very good.

TABLE 9.—EXPERIMENTS ON MODEL OF PIT NO. 4 DAM.
GROUP 5, TWO-LEVEL APRONS. (SEE FIG. 17.)

Experiment No.	h.	Remarks.	Stilling action.
38	11⅜ in.	....	Action good. Water thrown up by flat face of right splitter.
39	11⅜ in.	Right splitter reduced to half width.	Same as Experiment No. 38, except right splitter only threw thin sheet of water.
40	11⅜ in.	Right splitter removed.	Action very good except for water which washes up right wing-wall at bucket, as in all tests of this group.
41	11⅜ in.	Right splitter cut to fit against wing-wall.	Action same, except slightly more splash on right side.
42	5 in.	Right splitter as in Experiment No. 41. Splitter at step between apron levels increased to height of piers on high portions.	Reduced control did not effect action. Raising center pier caused better action.
43	5 in.	Same as Experiment No. 42, except piers spaced at 2 ft. 1¾ in.	Action not as good as previous experiments.
44	11⅜ in.	Same as Experiment No. 42.	Action better. Not as good as with closer pier spacing.

The experiments showed that the baffle-piers were by far the most effective destroyers of energy for such a deep over-pour as is proposed for Pit No. 4 Dam. In fact, none of the other layouts was satisfactory.

Of the arrangements of piers, the best was with a jump-off from the bucket of the dam, thus making the jet of water strike the piers at about mid-height With no jump-off, that is, with the overflow jet striking at the base of the piers, the water passes into the air in vertical sheets, unless there is considerable back-water from the channel below. With the jump-off the back-water, or lack of back-water, has little affect on the baffle-piers.

After a series of tests on a one-level apron a second series was undertaken with a two-level apron similar to that actually used in the design of the Pit No. 4 Dam. The model was accordingly altered to represent the two-level apron type. Results of tests of this layout and with slight modifications are shown in Fig. 11. The stilling action was not affected by the two level

feature. The action was just as satisfactory on the high apron side as if the whole apron was at that elevation. The "killing" of velocity was accomplished with less commotion on the deep side than on the shallow, but the difference in the final result, as indicated by the velocity in the river channel below, was, as nearly as could be judged, uniform across the width.

The flat up-stream face of the splitter pier near the right wing-wall seemed to throw considerable water into the air. Even with a sharp cutting-edge, this pier seemed to do more harm than good.

Incidentally, it was noted that the edge of the over-pour water at the right end of the dam maintained a vertical face until reaching the bucket curve where it flattened out with a resulting component of velocity along the length of the dam. This water impinged against the sloping right-bank wing-wall and washed up this wall to a height corresponding to 40 ft. on the actual structure. It is thought that this action could be eliminated either by a training wall on the down-stream face of the dam, or by moving the wing-wall out so as to be in line with the end of the spill section of the dam.

Observations on Completed Structure—Pit No. 4 Dam was completed after the high-water season of 1927. It will not be possible to observe the action of the piers in destroying the energy of the over-pour water under actual flood conditions until the spring of 1928.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

MAXIMUM FLOOD DISCHARGE IN SAN JOAQUIN VALLEY, CALIFORNIA

By Oren Reed,* Jun. Am. Soc. C. E.

Synopsis

Much has been written on the subject of floods in the last few years and many formulas have been presented for general or local application. Since the effect of similar storms on a given water-shed may vary widely, depending on temperature and storage conditions, formulas must be used with caution. In this paper an attempt has been made to estimate flood discharge for one river from the known discharges of rivers of the same meteorological province.

In each meteorological province there seems to be a type storm, the characteristics of which are relatively constant from time to time. Records of several streams in such a province, similar in storage, vegetation, soil, and other physical characteristics, can be combined, after the proper adjustments are made, to give an equivalent record for the stream under consideration. When long-term records are available each drainage area may be considered by itself, but records of many years are required to estimate the extreme flood to be expected on a stream. The method may be used to estimate flood discharge of a stream, where no flow measurements have been made, by comparison with the known discharges of neighboring water-sheds.

Flood Characteristics

In many localities stream-flow records are few or entirely lacking; then the engineer must rely on a study of the evidences of former floods, and on the conflicting stories of inhabitants of the neighborhood. As an aid in flood study many different formulas have been proposed for computing probable flood run-off. Some are only applicable to the small area for which

Note.—Written discussion on this paper will be closed in **March, 1928.**

* Asst. Designing Engr., San Joaquin Light & Power Corporation, Fresno, Calif.

they were originally devised on account of local conditions of soil, soil cover, slope of drainage, storage, etc. The use of most flood formulas is dangerous unless due regard is given to their basic limitations.

Except for small areas, the great floods in the Pacific Coast section occur in winter or early spring and are caused by heavy rainfall. On the larger streams, the drainage areas of which extend to high altitudes, the flood stage may be augmented, in some years, by the melting of snow.

In winter, an area of low barometric pressure exists over the North Pacific Ocean. As the great cyclonic storms sweep easterly or southeasterly from the Pacific, they yield gentle but persistent rainfall. After the storm center has passed and the wind swings from southeast to southwest, the rainfall increases in intensity for a short time; the winds then swing to the northwest and the storm has passed. In the early winter and late spring the storms follow each other in rapid succession, and heavy precipitation from these successive storms tends to cause extreme flood conditions. In the early part of winter, when a storm period that produces a considerable depth of snow is followed by a series of warm rains, a high flood is to be expected. If the snow becomes packed its water content is greatly increased, but high run-off will only result after several days of warm weather. With the rising temperature of spring, the dense snow cover starts to melt, augmented by short storms. This spring run-off seldom causes as high peaks as the mid-winter flood period, but gives a long-sustained stage of more moderate proportion.

Exceptions to these general rules are the spring-fed streams flowing from the lava country of Northern California, Eastern Oregon, and Southern Idaho, and small cloudburst areas, as in Oregon. Streams from the lava country are characterized by run-off of a marvelous constancy.

Flood Variation

While making an investigation to determine the probable maximum flood of the North Fork of Kings River, for the San Joaquin Light and Power Corporation, at Fresno, Calif., the writer was confronted with a maze of factors influencing the problem. The results of several formulas could not be reconciled, and it was decided to study previous floods on the Kings River at Sanger and on ten other major streams draining the west slope of the Sierra Nevada Mountains (Fig. 1), namely, the Cosumnes, Mokelumne, Stanislaus, Tuolumne, Merced, Fresno, San Joaquin, Kaweah, Tule, and Kern Rivers. Proper factors were determined to apply to a flood on one stream to make it applicable to Kings River.

To be strictly applicable to a particular drainage area, a formula for flood run-off must be modified to suit local conditions. General rainfall and run-off characteristics are relatively constant in each type province, but minor differences will be numerous. Small tributary streams may give widely different flood peaks on account of variable topographic conditions. Rancheria Creek, a tributary of the North Fork of Kings River, having a drainage area of about 25 sq. miles, is an example of low flood variation. The average elevation of the area is about 8 000, and the total run-off per square mile is high. However, the stream is not flashy. It drains a northwest slope, and

the forest cover of pine and spruce is heavy. Until the stream reaches the canyon of the North Fork of Kings River, the slope is moderate and the drainage is retarded by many lakes and meadows.

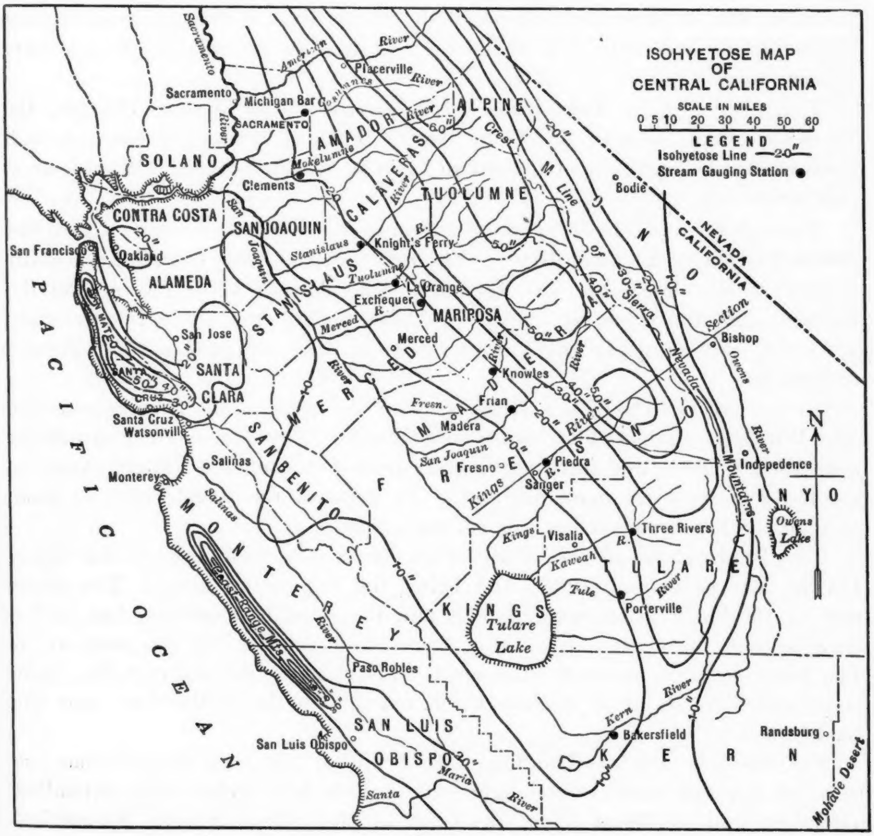

FIG. 1.

Physical Features

General.—The streams chosen for this investigation are tributary to the San Joaquin River or discharge into the Tulare and Kern Lake Basins and, together, drain the west slope of the Sierra Nevada Mountains and discharge into the southern half of the Great Valley of California. The Central Valley is a deep structural trough filled with alluvial material derived principally from the débris brought down by those streams from the Sierra Nevadas. The Sierras are a granitic formation with small areas of volcanic material. The streams rise at or near the divide and flow by a rapid gradient and in deep canyons to the outwash plain. The rainfall decreases toward the south, but the precipitation characteristics are similar on all eleven streams considered (Fig. 1).

Kern River.—Kern River, the most southern of the great rivers that flow westward from the crest of the Sierras, has a long, narrow basin. The general trend of the basin is north and south, except at the lower end. At the head-waters is the Mt. Whitney region, the highest and roughest in the Sierras. The main or North Fork of the Kern River is very rugged, whereas the South Fork is more flat in its lower basin. In general, the forest cover is very poor.

Tule River.—The Tule River heads on the Great Western Divide, the secondary ridge west of the Kern River. Part of the upper basin is well forested, but a large part of the entire basin is densely covered with chaparral and scrub oak.

Kaweah River.—The head-water region of the Kaweah is very rugged, but does not reach to the main divide. The ridges of the lower country, especially on the Marble Fork, are well forested. The flow of short rivers, like the Kaweah and the Tule, is decidedly flashy, owing to the extremely steep gradients, the relatively small drainage areas, and the predominant granite formations.

Kings River.—The Kings River rises at the crest of the Sierras near the Mt. Whitney group. The South and Middle Forks have steep gradients near their sources and have carved out profound canyons. Their slopes in the lower courses are more moderate. In general, the forest cover is good, although it has been cut over in certain areas.

The head-waters of the North Fork drain the west side of Le Conte Divide, Mt. Reinstein (12 575 ft.) being the highest elevation. The upper part of the basin is an open plateau and the slope is moderate, but in the lower course the stream drops at a tremendous gradient to the main river. The lower basin is covered with scrub timber; above Elevation 4 000, there is an excellent stand of timber which extends to the timber-line near the head-waters.

San Joaquin River.—The mountain basin of the San Joaquin has the form of a rough isosceles triangle with a base fifty miles long, extending northwest and southeast along the crest of the Sierra Nevada Mountains. The course of the main stream is, in general, southwest, while the principal tributaries near its head-waters flow through troughs trending parallel to the main divide. The brush-covered foothills give place to well-forested ridges, extending up about 9 500 ft., above which the cover thins out and the upper basin is entirely bare. Some storage has been developed in the upper part of the basin: Crane Valley Reservoir, 45 000 acre-ft.; Huntington Lake, 88 000 acre-ft.; and Florence Lake, 60 000 acre-ft.

Fresno River.—The Fresno River does not reach the main divide, but heads on the secondary ridges west of the San Joaquin River.

Merced River.—The Merced River rises at an elevation of about 11 000 ft. on the west slope of the secondary ridges, west of the head-waters of the Tuolumne and San Joaquin Rivers, its basin touching the main divide of the Sierras at only one point, Mt. Lyell (13 090 ft.). The topography of the head-water region is extremely rugged, but below Yosemite Valley it becomes

more regular and the stream is shut in by two high brush-covered ridges. Much of the middle basin, especially on the South Fork, is well forested. Exchequer Reservoir, constructed near the foothills, has a capacity of 280 000 acre-ft.

Tuolumne River.—The topography of the head-waters of the Tuolumne is very rugged. The Hetch Hetchy Valley, which is in the upper basin, rivals the Yosemite Valley in grandeur. Below Hetch Hetchy Valley the water-shed is more open. Most of the drainage area is well covered with forest interspersed with bare granite ridges and mountain meadows. Two reservoirs have recently been completed on the Tuolumne—Don Pedro, near the foothills, 270 000 acre-ft., completed in 1923, and O'Shaughnessy Reservoir, 206 000 acre-ft., at the lower end of Hetch Hetchy Valley.

Stanislaus River.—The Upper Stanislaus Basin is typically Alpine, including tremendous domes and peaks of granite and very rugged topography. In the lower course the stream is shut in by two broad-topped ridges, which are covered with heavy timber. Near the head-waters of the basin there are several small storage reservoirs; Relief Reservoir, 14 965 acre-ft., and Strawberry Reservoir, 17 900 acre-ft., are the largest.

Mokelumne River.—The Mokelumne Basin is long and comparatively narrow, and extends nearly due west from the crest of the Sierras to the eastern edge of the foothills. Round Top (10 430 ft.), is the highest peak. Except in the higher regions, the country presents a spacious and rolling, rather than rugged, appearance. The basin is well forested, except near the head-waters which is for the most part of bare granite.

Cosumnes River.—Like the Mokelumne River, the Cosumnes Basin is long and narrow; for a great part of their length the tributaries flow in nearly parallel canyons. The highest peak is Alder Hill (7 875 ft.).

Precipitation and Seasonal Run-Off

As an aid in interpreting flood data, a study was made of seasonal precipitation and run-off. An isohyetose map of the south-central part of the State was drawn (Fig. 1).* Records of mean precipitation of 114 stations were used. The number of years included in the record at each station varied from 54 at Stockton and Santa Barbara to only 4 for the stations established by the San Joaquin Light and Power Corporation on the North Fork of Kings River. Four, or even ten, years are too few to determine true averages, but some short-term records were used to show the general trend. The records of the stations on the North Fork of Kings River were adjusted somewhat by a comparison with the records of the corresponding years at Huntington Lake on the adjoining San Joaquin water-shed.

Except for thunder-storms, which occur in the mountains in the late summer and early fall, precipitation on the west slope of the Sierras results

* The base for this map is the General Land Office Map of California. Isohyetose lines are based on precipitation as given for various stations in *Bulletin No. 5*, California Department of Public Works, except those for San Francisco Bay region, which are as given by C. E. Grunsky, Past-President, Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 104.

from the movement of great cyclonic storms. The general track of these storms is from northwest to southeast. They enter the State from the Pacific Ocean near the northwest corner and, after crossing the Coast Range, follow a path roughly parallel to the trend of the Sierras. Drainage areas near the north end of the area considered—the Stanislaus and the Mokelumne—receive heavy precipitation. As the storm passes southward the atmosphere becomes dryer and beyond the Kings River Basin the decrease in precipitation is rapid. The southern water-sheds are rarely visited by cyclonic storms from the southwest. The variation in precipitation from north to south is shown by Fig. 2, which is based on three groups of stations, each group being in a definite elevation belt. During a wet winter, as in 1916, the storm center is shifted southward and the Kern and Tule Rivers receive much heavier precipitation.

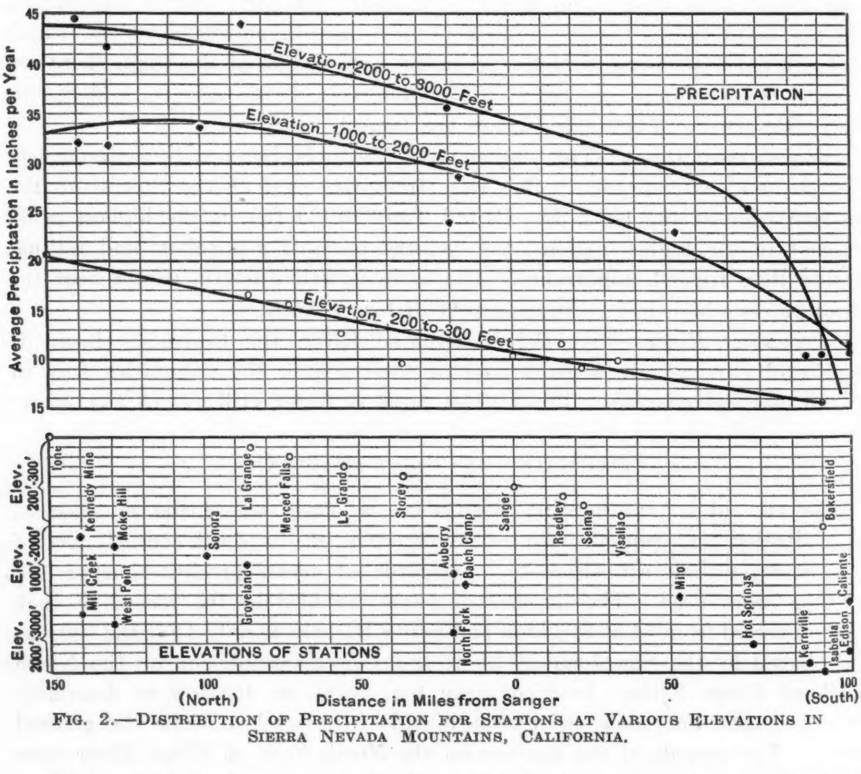

FIG. 2.—DISTRIBUTION OF PRECIPITATION FOR STATIONS AT VARIOUS ELEVATIONS IN SIERRA NEVADA MOUNTAINS, CALIFORNIA.

When precipitation records on a section at right angles to the trend of the Sierras are studied, it is seen that, to a certain elevation, precipitation increases with height. Beyond this, there is a slow decrease to the summit and a rapid decrease east of the mountains (Fig. 3). This Kings River Section is indicated on Fig. 1. The curves plotted for this section support the conclusions* of C. H. Lee, M. Am. Soc. C. E., that topography rather

* "The Determination of Safe Yield of Underground Reservoirs of the Closed-Basin Type," *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 148.

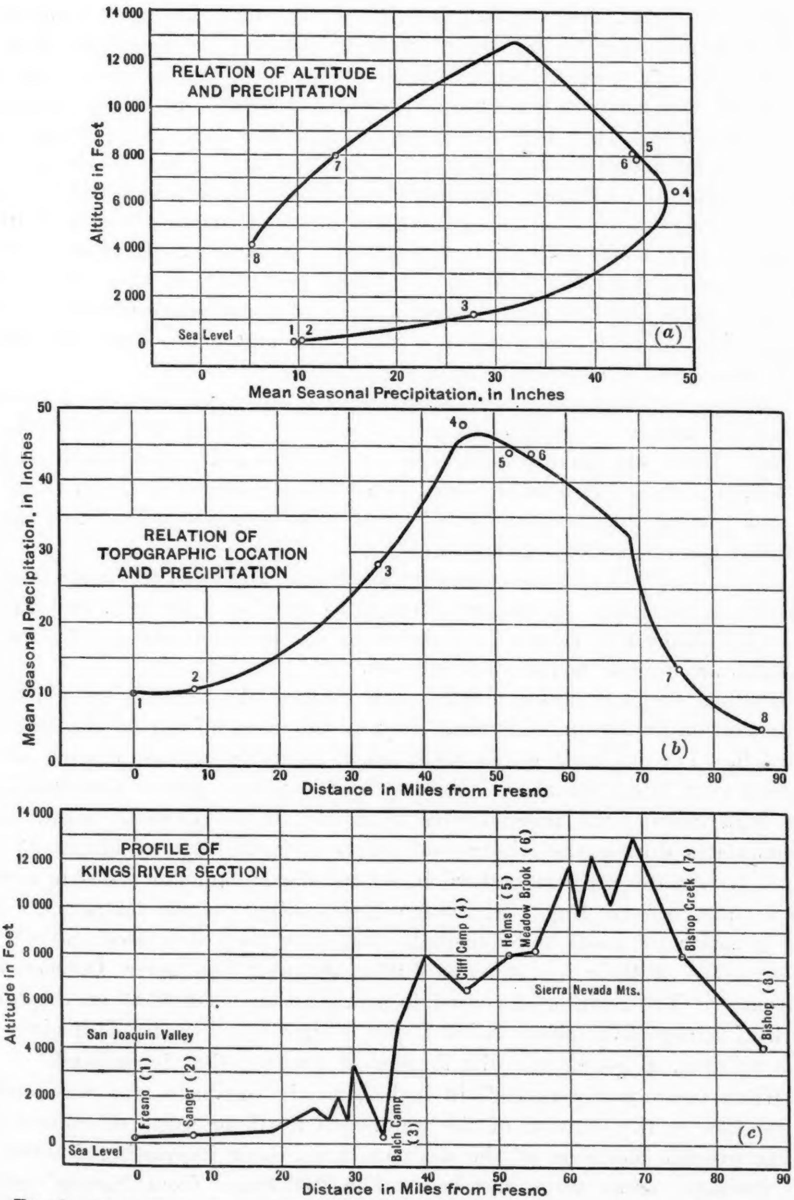

FIG. 3.—INFLUENCE OF ALTITUDE AND TOPOGRAPHIC LOCATION ON PRECIPITATION.

than elevation is the controlling factor in the distribution of total precipitation in the Sierra Nevada Mountains.

The greatest rainfall occurs at an elevation of 4 000 to 6 000 ft., differing with each storm, due to variation in relative humidity and temperature. The moisture-laden winds, which are blown against the mountain slope, are forced upward, giving rise to rapid cooling and precipitation. Heat is liberated with condensation and, as a result, the dew-point of the atmosphere is raised above this belt of maximum precipitation. The precipitation decreases gradually to the summit of the range, and as the winds pass over the divide, their absolute humidity having been decreased by the induced rainfall, they are compressed in their flow down the mountain with a further increase in temperature and, in consequence, become dry. If, after the winds pass through the lower area of maximum precipitation, they flow over a plateau region and lose heat, a second range of mountains will give rise to a second belt of high precipitation. Mountains also cause local atmospheric circulation and consequent orographic rainfall.

Although a large proportion of the precipitation in this area is brought about by general storms, sometimes severe storms resembling cloudbursts occur. There was an unusual storm at Fresno on October 4 and 5, 1925. A total rainfall of 1.56 in. occurred in a 36-hour period; of this 1.18 in. fell in one violent thunder-storm of 32 min. The area covered by this intense rainfall was only a few square miles.

It was observed that the precipitation characteristics of the twelve streams studied are similar. Secondary streams, as the Tule, Kaweah, and Fresno, are more flashy and have a long period of low flow, but the general characteristics are similar to the other streams.

During the late autumn and early winter the foothill area receives precipitation in the form of rain. Most of this runs off during the season of rainfall. The outwash slopes yield no appreciable surface run-off, except during very heavy storms, on account of the porous gravel formation. In the high Sierras the precipitation is chiefly in the form of snow. This accumulates during the winter and starts to melt between March 15 and April 1. As the air temperature increases, the stream flow rises to a peak some time between May 1 and June 1, depending on the temperature and the quantity of snow to be melted. The flow then decreases and all the streams are at their lowest stage during August, September, October, and November. The relation of run-off to precipitation for 1922-23 on the North Fork of Kings River above Balch Camp is shown in Fig. 4. This gives the true relation, although the distribution of precipitation is unusual.

When mean yearly run-off, in inches, on the drainage area was studied in relation to the location of the water-shed north or south of Sanger and to the average elevation of the drainage area, some interesting features of the drainage areas were observed. The "Distance from Sanger" curve (Fig. 5) was similar to Fig. 2, as was expected, but the low yield of the southern streams, the Kern and the Tule, was very apparent. Precipitation

stations on many of the streams are in the foothills and their records are a poor basis for determining the average precipitation on the drainage basins as a whole. The Fresno is a secondary stream with no tributary area of high elevation. The highest elevation of the water-shed is 6 350 ft. The relation of run-off to area of drainage (Fig. 6) showed more clearly the deficiency of run-off of the Kern, Tule, and Fresno.

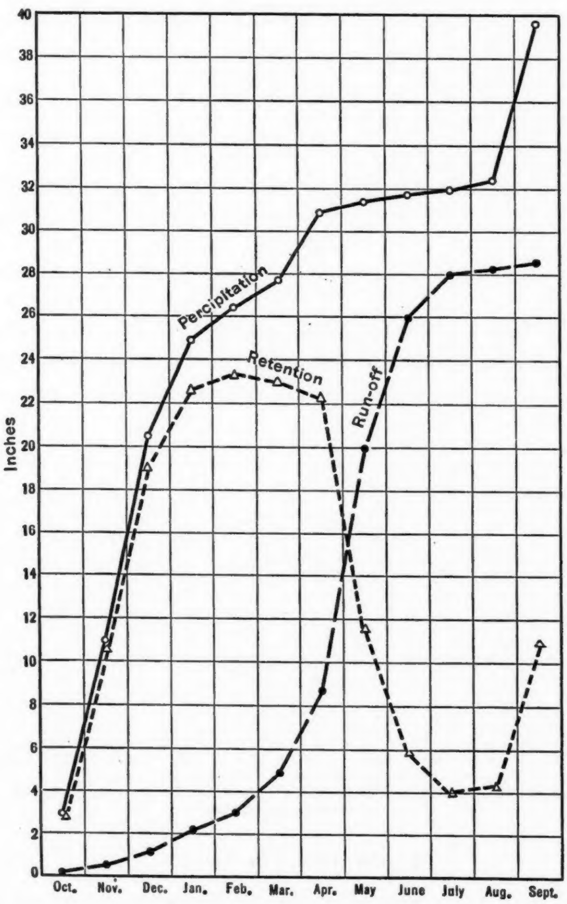

FIG. 4.—MASS DIAGRAMS OF RUN-OFF. PRECIPITATION RELATIONS ON NORTH FORK OF KINGS RIVER AT BALCH CAMP, 1922–23, SLIGHTLY BELOW NORMAL.

FLOOD DATA

The flood data used in this paper were taken from the *Water Supply Papers* of the U. S. Geological Survey. The primary data for the eleven California streams are given in Tables 1 and 2.

Gauging stations have been established by the U. S. Geological Survey on each of the rivers under consideration near the point where the stream leaves the foothills. The longest published record is for the Kings River near

Sanger, 26 years; the shortest record is for the Fresno, 8 years. The Kings River Gauging Station is at Piedra, a few miles northeast of Sanger. Until the seasonal year, 1913-14, only the maximum 24-hour flood was published.

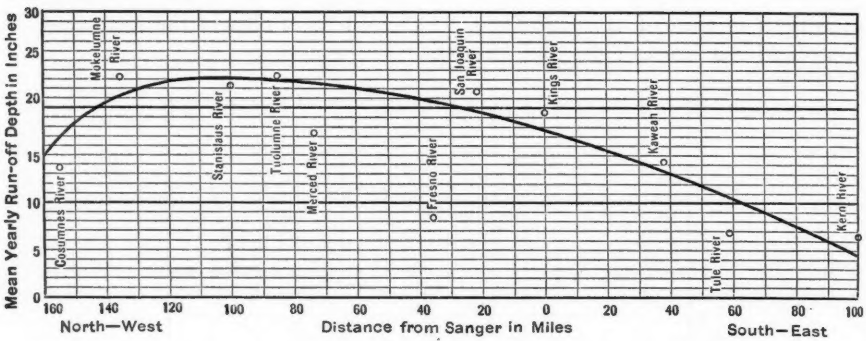

FIG. 5.—RELATION OF YEARLY RUN-OFF TO GEOGRAPHICAL LOCATION OF ELEVEN CALIFORNIA STREAMS.

In Table 1 the "24-hour flood corresponding to the peak flood" is given. In some years this peak flood did not occur on the same day as the maximum 24-hour flood, but came as a sudden and unsustained peak during the winter.

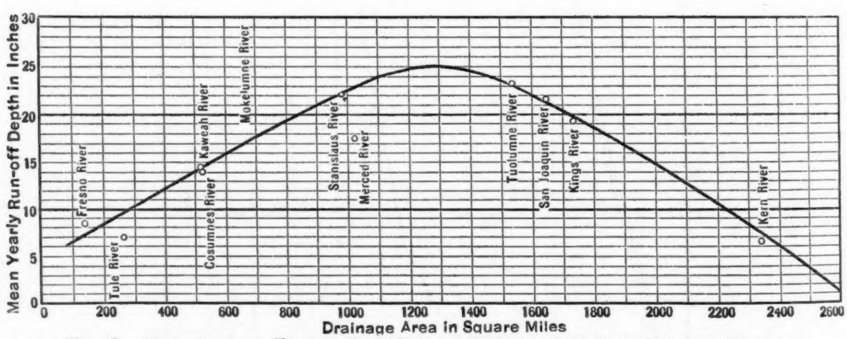

FIG. 6.—RELATION OF YEARLY RUN-OFF TO SIZE OF DRAINAGE AREA OF ELEVEN CALIFORNIA STREAMS.

FLOOD DETERMINATION

If sufficient data were available, the most accurate way of determining the maximum flood on any stream would be to consider each stream by itself. A record of at least 100 years on each stream would be necessary, in order to establish probable variations of flood run-off. Even that length of record might be inadequate, for maximum flows are often grouped close together with a long interval of medium, or low, flows. Flood records of the Danube River at Vienna, Austria, which cover 900 years, show that a record of 100 years is not sufficient to fix the maximum flood. Three extraordinary floods have occurred at Vienna with time intervals of 112 and 286 years. The greatest flood was one-third greater than the second record flood.

TABLE 1.—FLOOD DATA FOR CALIFORNIA STREAMS.

Seasonal year.	KERN RIVER, AT BAKERSFIELD, CALIF. MEAN 24-HOUR FLOOD, 5 093 SEC-FT.					TULE RIVER, NEAR PORTERVILLE, CALIF. MEAN 24-HOUR FLOOD, 2 556 SEC-FT.					KAWEAH RIVER, NEAR THREE RIVERS, CALIF. MEAN 24-HOUR FLOOD, 4 683 SEC-FT.				
	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.
1895-1896	......	....	......	......	....	......	....	......	......	....	......	....	......	......	....
1896- 97	5 342	103	......	......	....	......	....	......	......	....	......	....	......	......	....
1897- 98	1 342	26	......	......	....	......	....	......	......	....	......	....	......	......	....
1898- 99	4 932	97	......	......	....	......	....	......	......	....	......	....	......	......	....
1899-1900	1 852	36	......	......	....	......	....	......	......	....	......	....	......	......	....
1900- 01	4 295	85	......	......	....	......	....	......	......	....	......	....	......	......	....
1901- 02	3 758	74	......	......	....	4 615	180	......	......	....	......	....	......	......	....
1902- 03	3 374	66	......	......	....	3 790	148	......	......	....	......	....	......	......	....
1903- 04	3 167	62	......	......	....	2 305	90	......	......	....	2 920	63	......	......	....
1904- 05	3 039	60	......	......	....	822	32	......	......	....	2 730	58	......	......	....
1905- 06	9 505	186	......	......	....	4 530	177	......	......	....	8 160	174	......	......	....
1906- 07	......	....	......	......	....	2 300	91	......	......	....	3 400	73	......	......	....
1907- 08	......	....	......	......	....	1 230	48	......	......	....	1 600	34	......	......	....
1908- 09	8 851	174	......	......	....	5 070	199	......	......	....	9 210	197	......	......	....
1909- 10	4 656	92	......	......	....	5 430	212	......	......	....	7 910	170	......	......	....
1910- 11	4 623	91	......	......	....	2 780	109	......	......	....	6 610	141	......	......	....
1911- 12	2 919	57	......	......	....	260	10	......	......	....	2 360	50	......	......	....
1912- 13	1 976	39	......	......	....	248	10	......	......	....	1 470	31	......	......	....
1913- 14	15 478	305	15 478	18 287	118	4 710	184	4 710	6 600	140	9 880	211	9 880	13 300	135
1914- 15	4 010	79	4 010	4 249	106	1 380	51	1 380	1 740	126	3 280	70	3 280	3 600	110
1915- 16	16 125	317	16 125	17 962	111	4 080	160	4 080	6 780	166	10 100	216	10 100	14 700	146
1916- 17	3 976	78	3 976	4 331	109	3 540	139	3 540	4 260	120	3 280	70	2 690	6 270	233
1917- 18	3 162	62	3 162	3 478	110	665	26	427	900	211	1 400	30	1 400	1 570	112
1918- 19	3 851	76	3 851	4 299	111	665	26	500	1 070	214	2 690	57	2 690	2 830	105
1919- 20	3 657	72	3 449	4 456	129	2 140	84	2 140	3 450	161	4 850	103	4 850	5 850	121
1920- 21	3 247	64	3 247	3 548	109	565	22	565	657	116	2 450	52	2 450	3 190	130

TABLE 1.—(*Continued*).

Seasonal year.	KINGS RIVER, NEAR SANGER, CALIF. MEAN 24-HOUR FLOOD, 17 358 SEC-FT.					SAN JOAQUIN RIVER, NEAR FRIANT, CALIF. MEAN 24-HOUR FLOOD, 17 118 SEC-FT.					FRESNO RIVER, NEAR KNOWLES, CALIF. MEAN 24-HOUR FLOOD, 1 807 SEC-FT.				
	Maximum 24 hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.
1895–1896	22 100	127	......	......	....	......	....	(1 880)	60 000	....	......	....	......	......	....
1896– 97	22 732	131	......	......	....	......	....	......	......	....	......	....	......	......	....
1897– 98	8 348	48	......	......	....	......	....	......	......	....	......	....	......	......	....
1898– 99	20 200	116	......	......	....	......	....	......	......	....	......	....	......	......	....
1899–1900	12 700	73	......	......	....	......	....	......	......	....	......	....	......	......	....
1900– 01	43 930	253	......	......	....	......	....	......	......	....	......	....	......	......	....
1901– 02	26 380	152	......	......	....	......	....	......	......	....	......	....	......	......	....
1902– 03	17 290	99	......	......	....	......	....	......	......	....	......	....	......	......	....
1903– 04	15 700	91	......	......	....	......	....	......	......	....	......	....	......	......	....
1904– 05	9 795	56	......	......	....	......	....	......	......	....	......	....	......	......	....
1905– 06	26 600	153	......	......	....	......	....	......	......	....	......	....	......	......	....
1906– 07	15 600	90	......	......	....	......	....	......	......	....	......	....	......	......	....
1907– 08	6 920	40	......	......	....	6 130	36	......	......	....	......	....	......	......	....
1908– 09	20 300	117	......	......	....	28 800	168	......	......	....	......	....	......	......	....
1909– 10	14 700	85	......	......	....	27 900	163	......	......	....	......	....	......	......	....
1910– 11	20 500	118	......	......	....	38 800	226	......	......	....	......	....	......	......	....
1911– 12	12 400	71	......	......	....	15 300	89	......	......	....	189	10	......	......	....
1912– 13	7 210	41	......	......	....	6 610	39	......	......	....	118	7	......	......	....
1913– 14	30 400	176	30 400	59 700	196	24 700	144	24 700	46 200	187	......	....	......	......	....
1914– 15	16 300	94	16 300	18 300	112	13 800	81	13 800	15 900	115	......	....	......	......	....
1915– 16	16 300	94	14 200	45 400	320	13 200	77	10 700	17 000	159	3 770	208	2 570	4 010	156
1916– 17	13 200	76	9 400	17 900	190	13 400	78	9 900	15 600	158	3 770	208	3 770	4 500	119
1917– 18	12 800	74	12 800	13 500	106	......	....	......	......	....	2 490	138	2 490	2 710	109
1918– 19	11 200	65	11 200	13 200	118	10 900	64	......	......	....	1 740	96	1 740	1 940	111
1919– 20	14 900	86	14 900	17 000	114	11 500	67	11 500	14 000	122	942	52	530	970	183
1920– 21	12 800	74	12 400	16 000	129	11 500	67	11 500	15 300	133	1 440	80	1 440	1 740	121

TABLE 1.—(*Continued*).

Seasonal year.	MERCED RIVER, NEAR MERCED FALLS, CALIF. AFTER OCTOBER 1915, NEAR EXCHEQUER, CALIF. MEAN 24-HOUR FLOOD. 13 178 SEC-FT.					TUOLUMNE RIVER, AT LA GRANGE, CALIF. MEAN 24-HOUR FLOOD, 19 329 SEC-FT.					STANISLAUS RIVER, AT KNIGHT'S FERRY, CALIF. MEAN 24-HOUR FLOOD, 13 903 SEC-FT.				
	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.
1895–1896	......	....	......	......	....	11 798	61	......	......	....	......	....	......	......	....
1896– 97	......	....	......	......	....	14 700	76	......	......	....	......	....	......	......	....
1897– 98	......	....	......	......	....	7 800	41	......	......	....	......	....	......	......	....
1898– 99	......	....	......	......	....	21 800	113	......	......	....	......	....	......	......	....
1899–1900	......	....	......	......	....	18 160	68	......	......	....	......	....	......	......	....
1900– 01	......	....	......	......	....	19 240	100	......	......	....	......	....	......	......	....
1901– 02	6 240	47	......	......	....	12 934	67	......	......	....	......	....	......	......	....
1902– 03	11 400	87	......	......	....	20 342	105	......	......	....	......	....	......	......	....
1903– 04	8 780	67	......	......	....	17 850	92	......	......	....	......	....	......	......	....
1904– 05	8 020	61	......	......	....	15 855	82	......	......	....	......	....	......	......	....
1905– 06	18 400	140	......	......	....	24 400	126	......	......	....	......	....	......	......	....
1906– 07	27 500	209	......	......	....	52 000	270	......	......	....	......	....	......	......	....
1907– 08	3 740	28	......	......	....	6 720	35	......	......	....	......	....	......	......	....
1908– 09	19 500	148	......	......	....	26 700	138	......	......	....	36 600	263	......	......	....
1909– 10	14 800	112	......	......	....	20 900	108	......	......	....	9 830	71	......	......	....
1910– 11	37 200	282	......	......	....	52 600	273	52 600	60 300	115	36 900	266	36 900	60 200	163
1911– 12	6 100	46	......	......	....	13 800	72	......	......	....	5 880	42	......	......	....
1912– 13	3 130	24	......	......	....	6 320	33	......	......	....	4 380	31	......	......	....
1913– 14	......	....	......	......	....	31 200	161	......	......	....	16 200	116	16 200	32 200	199
1914– 15	......	....	......	......	....	13 300	69	13 300	17 200	129	9 310	67	9 310	11 100	119
1915– 16	12 600	96	7 240	22 000	304	15 400	80	15 400	20 500	133	11 200	81	11 200	14 200	127
1916– 17	18 500	140	17 400	21 700	125	22 900	119	22 900	36 500	159	13 200	95	13 200	17 400	132
1917– 18	13 200	100	10 500	17 700	169	15 100	79	15 100	23 300	154	11 800	85	11 800	14 300	121
1918– 19	7 740	59	7 740	8 540	110	13 500	70	13 500	15 600	116	7 740	56	7 740	9 700	125
1919– 20	7 360	56	6 320	8 950	142	12 900	68	11 000	15 500	141	6 800	49	6 800	8 860	130
1920– 21	13 000	99	13 000	19 400	149	......	....	......	......	....	10 900	78	10 900	16 200	149

TABLE 1.—(*Continued*).

Seasonal year.	MOKELUMNE RIVER, NEAR CLEMENTS, CALIF. MEAN 24-HOUR FLOOD, 8 036 SEC-FT.					COSUMNES RIVER, NEAR MICHIGAN BAR, CALIF. MEAN 24-HOUR FLOOD, 9 696 SEC-FT.				
	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.	Maximum 24-hour flood.	Percentage of variation from mean.	24-hour flood corresponding to peak.	Peak flood.	Percentage, peak to 24-hour flood.
1895-1896	....	....	....	....	....	....	....	....	....	....
1896- 97	....	....	....	....	....	....	....	....	....	....
1897- 98	....	....	....	....	....	....	....	....	....	....
1898- 99	....	....	....	....	....	....	....	....	....	....
1899-1900	....	....	....	....	....	....	....	....	....	....
1900- 01	....	....	....	....	....	....	....	....	....	....
1901- 02	....	....	....	....	....	....	....	....	....	....
1902- 03	....	....	....	....	....	....	....	....	....	....
1903- 04	....	....	....	....	....	....	....	....	....	....
1904- 05	5 260	65	....	....	....	....	....	....	....	....
1905- 06	8 740	108	....	....	....	....	....	....	....	....
1906- 07	15 300	191	....	....	....	....	....	....	....	....
1907- 08	2 960	37	....	....	....	2 180	23	....	....	....
1908- 09	10 400	130	....	....	....	20 800	216	....	....	....
1909- 10	7 200	90	....	....	....	7 200	74	....	....	....
1910- 11	16 700	208	16 700	20 500	123	22 400	232	....	....	....
1911- 12	4 920	61	....	....	....	1 100	11	....	....	....
1912- 13	3 840	48	....	....	....	1 220	13	....	....	....
1913- 14	11 100	139	11 100	13 300	120	13 900	144	13 900	18 200	131
1914- 15	7 750	97	6 440	8 290	129	5 920	61	5 920	8 200	138
1915- 16	8 040	100	8 040	10 700	133	8 920	92	8 920	10 400	117
1916- 17	7 550	94	....	....	....	13 500	140	13 500	22 900	170
1917- 18	6 940	86	6 940	8 620	124	10 800	111	10 800	11 900	110
1918- 19	7 060	88	7 060	7 540	107	13 100	135	13 100	22 000	168
1919- 20	5 500	69	5 500	6 820	124	3 210	33	3 210	3 700	115
1920- 21	7 350	91	7 350	11 100	150	11 500	119	11 500	20 600	179

TABLE 2.—COMPARISON OF GEOGRAPHY AND DRAINAGE FOR CALIFORNIA STREAMS.

	Kern River, near Bakersfield, Calif.	Tule River, near Porterville, Calif.	Kaweah River, near Three Rivers, Calif.	Kings River, near Sanger, Calif.	San Joaquin River, near Friant, Calif.	Fresno River, near Knowles, Calif.	Merced River, near Exchequer, Calif.	Tuolumne River, near La Grange, Calif.	Stanislaus River, near Knight's Ferry, Calif.	Mokelumne River, near Clements, Calif.	Cosumnes River, near Michigan Bar, Calif.
Drainage area, in square miles	2 345	266	520	1 740	1 640	134.4	1 020	1 542	987	631	525
Average elevation, in feet	6 780	4 185	5 805	7 195	7 050	3 215	5 570	5 892	5 055	5 020	3 200
Distance from Sanger, in miles	100 (S.)	58 (S.)	38 (S.)	......	22 (N.)	36 (N.)	73 (N.)	85 (N.)	100 (N.)	135 (N.)	155 (N.)
Maximum 24-hour flood, in second-feet	16 125	5 430	10 100	43 930	38 800	3 770	37 200	52 600	36 900	16 700	22 400
Second-feet per square mile	6.88	20.41	19.42	25.25	23.66	28.07	36.47	34.11	37.39	26.47	42.67
Mean 24-hour flood, in second-feet	5 093	2 556	4 683	17 358	17 118	1 807	13 178	19 329	13 903	8 036	9 696
Second-feet per square mile	2.17	9.61	9.01	9.98	10.44	13.45	12.92	12.54	14.09	12.74	18.47

Keeping in mind the relation of mean run-off on the eleven streams, a maximum 24-hour flood was determined for the Kings River, at Sanger, which corresponded to the highest 24-hour flow on each stream in turn. The fact that the flood flow on each stream was the maximum for a period varying from 8 to 26 years had the effect of giving many comparable data for the Kings River at Sanger. Although the maximum rate of run-off is usually more important than the average for 24 hours, the 24-hour flood records were used because they were greater in number.

When the maximum 24-hour flood, in second-feet per square mile, for each stream was plotted according to its geographic location northwest or southeast of the Kings River (Fig. 7), a variation was shown similar to that for precipitation (Fig. 2). Flood flows decreased rapidly to the southward. The relative humidity of a storm, as it travels southward, becomes less, and the Kern Basin receives only slight precipitation from storms that cause intense run-offs on other streams farther north. The highest flows on the Kern River are usually caused by melting snow in May and June.

Geographic location is also of major importance in the other causes of flood variations that were considered. The average elevation receiving the heaviest precipitation is shown by Fig. 8. It is seen to be about 5 500 ft. The Cosumnes River has an average elevation of only 3 200 ft. but as it is at the north end of the area studied, it receives a high precipitation, which produces an extreme flood run-off. The Tule River, on the other hand, is of greater average elevation, but as it is near the south end of the valley the precipitation and flood yield is much smaller.

In relation to size of drainage area small floods were produced by the small secondary streams, the Fresno and the Tule, and by the Kern, a primary stream (Fig. 9).

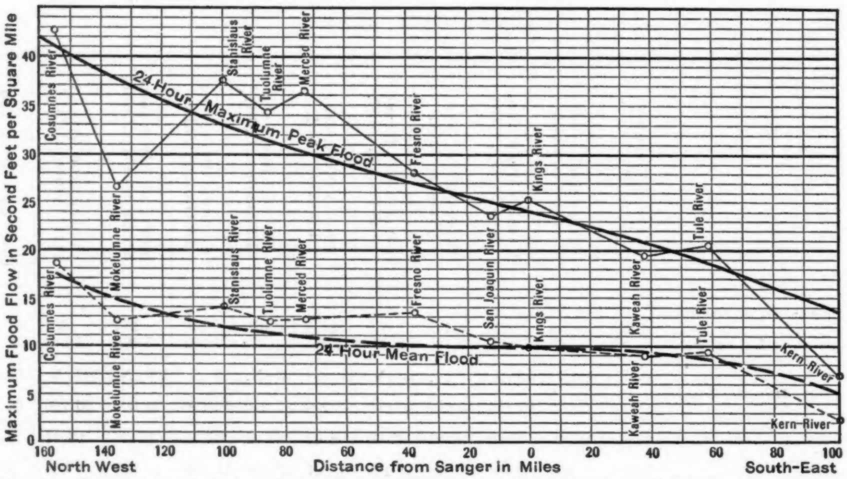

FIG. 7.—RELATION OF FLOOD RUN-OFF (24-HOUR) TO GEOGRAPHICAL LOCATION OF ELEVEN CALIFORNIA STREAMS.

In addition to the three factors of geographic location, average elevation, and drainage area, a fourth factor, the variation in proportion of the maximum 24-hour flood to the arithmetical mean, was used. This factor represents the susceptibility of a drainage area to floods or excessive rains. When a basin has a low variation factor the high stage will usually be caused by the sustained flows of melting snow in early summer; this is comparatively constant from year to year. High variations result from heavy storms during the early part of winter. Other factors, such as geologic formation, general topography, shape of the drainage basins, etc., are in this case of minor importance for, as has been pointed out, these physical features are similar among drainage basins in this area.

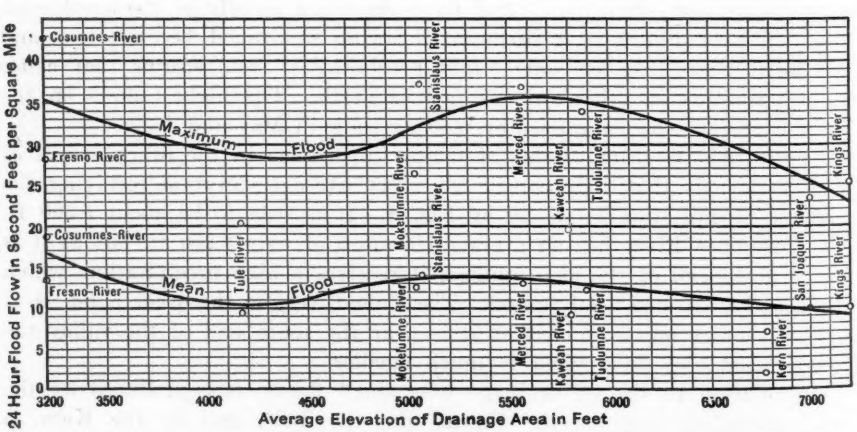

FIG. 8.—RELATION OF FLOOD RUN-OFF (24-HOUR) TO AVERAGE ELEVATION OF DRAINAGE AREA OF ELEVEN CALIFORNIA STREAMS.

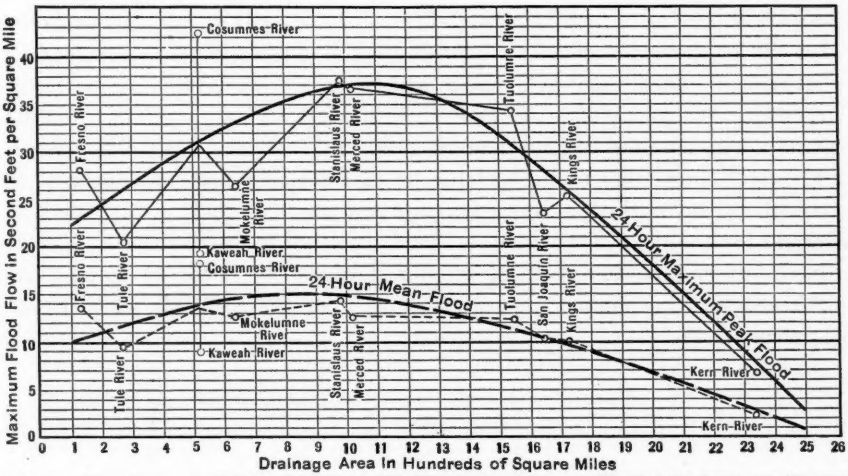

FIG. 9.—RELATION OF FLOOD RUN-OFF (24-HOUR) TO SIZE OF DRAINAGE AREA OF ELEVEN CALIFORNIA STREAMS.

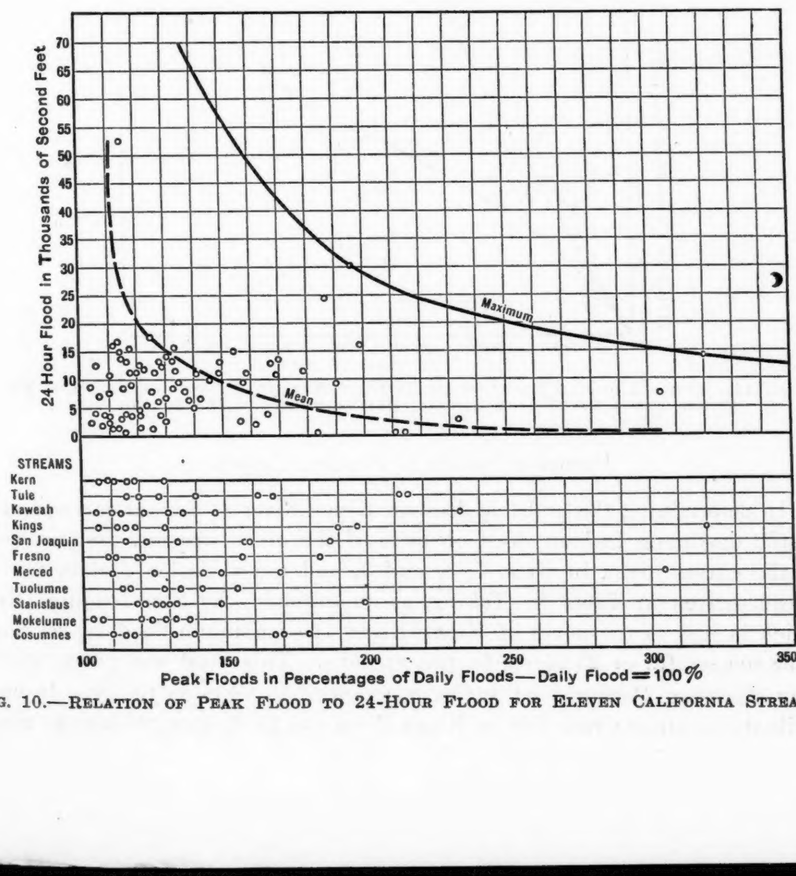

FIG. 10.—RELATION OF PEAK FLOOD TO 24-HOUR FLOOD FOR ELEVEN CALIFORNIA STREAMS.

Relation of Peak Flood to 24-Hour Flood

Peak floods have been published for the streams under consideration only since 1914, giving a total of 69 observations for peak floods in comparison with 186 observations of 24-hour floods on the eleven streams. Comparative relations of one stream to another could best be established by using the larger number of observations. Peak flood run-off is an inverse function of the corresponding 24-hour run-off (Fig. 10). When the floods occur in the early summer due to the melting of the snow cover on the upper watershed, the peak or diurnal variation is approximately 25% greater than the average for the day. On the other hand, peak floods in winter are less sustained, but may be 200 or 300% greater than the mean for 24 hours. This difference between summer and winter floods for the Kings River at Sanger is shown in Fig. 11.

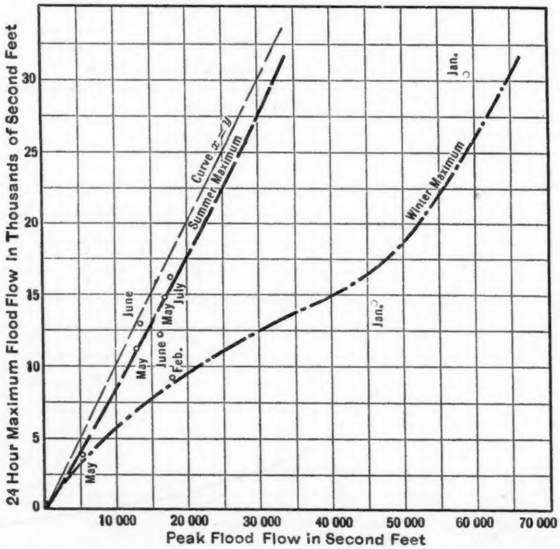

Fig. 11.—Curve Showing Relation of 24-Hour Maximum Flood to Maximum Peak Flood, Kings River near Sanger, Drainage Area, 1 740 Square Miles.

Probable Floods at Sanger, California

In determining the probable flood on Kings River at Sanger corresponding to the maximum 24-hour flood on each of the other streams, use was made of the curves given in Figs. 7, 8, and 9, and the variation factors for the streams given in Table 1. Taking, as an example, the San Joaquin River, which is just to the north of Kings River, the maximum flood for 24 hours is 38 800 sec-ft., or 23.66 sec-ft. per sq. mile. This flood was 226% greater than the mean 24-hour flood during a period of 13 years on the San Joaquin, while the maximum variation on Kings River was 253% greater than the mean.

In addition:

From Fig. 7, Correction for "Distance from Sanger":

San Joaquin River = 25.6 sec-ft. per sq. mile.
Kings River = 23.9 sec-ft. per sq. mile.
Correction = 93.3%

From Fig. 8, Correction for "Average Elevation of Drainage Area":

San Joaquin River = 25.8 sec-ft. per sq. mile.
Kings River = 23.2 sec-ft. per sq. mile.
Correction = 90.0%

From Fig. 9, Correction for "Drainage Area in Square Miles":

San Joaquin River = 28.0 sec-ft. per sq. mile.
Kings River = 25.0 sec-ft. per sq. mile.
Correction = 89.3%

The maximum probable 24-hour flood at Sanger is,

$$23.66 \times \frac{253}{226} \times 93.3\% \times 90.0\% \times 89.3\% = 19.85 \text{ sec-ft. per sq. mile,}$$

or, 34 550 sec-ft.

From Fig. 10, a peak flood of 64 200 sec-ft., or 36.90 sec-ft. per sq. mile, would be possible from a 24-hour flood of 34 550 sec-ft.

The other ten streams were studied in a like manner and gave the results shown in Table 3.

TABLE 3.—Deduced Flood Flow for Kings River near Sanger, California.

Stream.	24-hour flood, in second-feet per square mile.	Corresponding 24-hour flood, at Sanger, in second-feet per square mile.	Peak Flood, Kings River at Sanger.	
			Second-feet per square mile.	Second-feet.
Kern	6.88	25.10	42.6	74 100
Tule	20.41	25.25	42.6	74 200
Kaweah	19.42	14.28	31.5	54 900
Kings	25.25	25.25	42.2	73 300
San Joaquin	23.66	19.85	36.9	64 200
Fresno	28.07	22.10	41.4	72 100
Merced	36.47	11.77	30.0	52 200
Tuolumne	34.11	13.58	32.0	55 500
Stanislaus	37.39	13.20	31.1	54 100
Mokelumne	26.47	11.83	30.2	52 500
Cosumnes	42.67	15.13	33.1	57 500
Average	.....	.....	35.78	62 240

The greatest variations from the mean were the adjusted floods from the Kern and Tule Rivers, but the maximum variation is less than 20 per cent. The average value of the peak flood for Kings River, at Sanger, appears to be reasonable. It is 42% greater than the 29-hour flood of 1901 and only 4% greater than the peak flood of 1914.

As a check, a maximum probable flood was computed for each of the other ten streams in the same manner as for the Kings River. In each case the computed flood was reasonable.

Probable Floods at Balch Camp, California

The original intention of this investigation being to determine the probable maximum flood on the North Fork of Kings River, studies were made to determine the probable flow at Balch Camp at the confluence of the North and West Forks of Kings River for a given flow at Sanger on the main river. The San Joaquin Light and Power Corporation has planned a series of developments on the North Fork of Kings River (Fig. 12). The first stage of the Balch Project has been constructed. An arched diversion dam was built at Williams Crossing which will have an ultimate crest elevation of 4 110 ft. The water is then led by a 19 500-ft. tunnel along the north side of the stream to the penstock. Owing to the steepness of the canyon there was no feasible power-house site on the north side of the river, and the penstock will be carried over the river to the station on the opposite side.

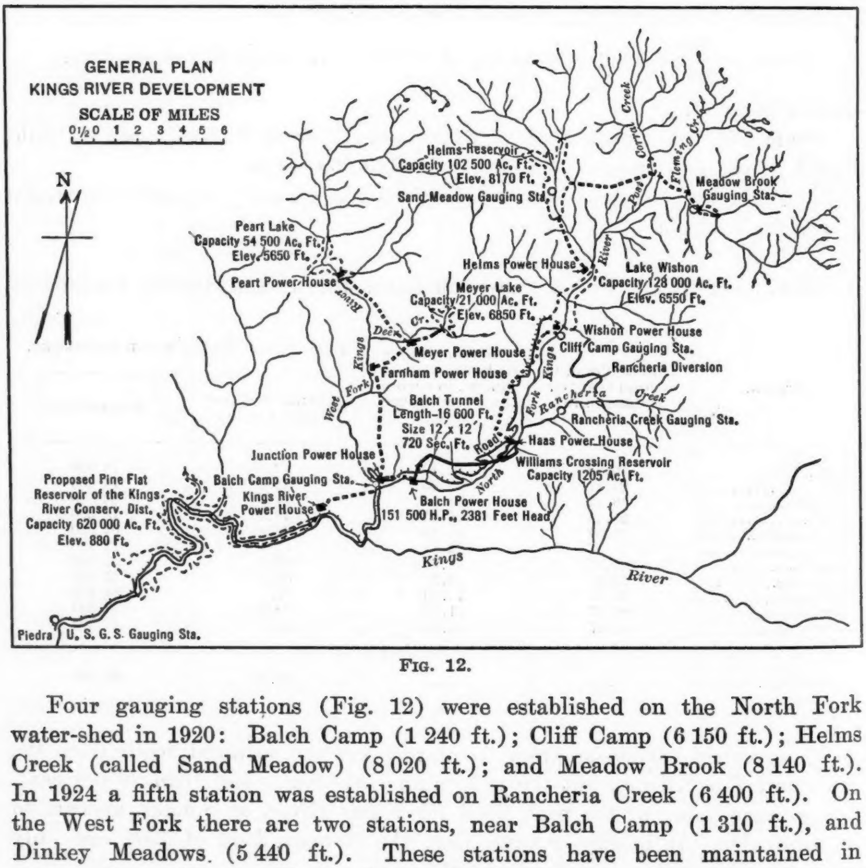

Fig. 12.

Four gauging stations (Fig. 12) were established on the North Fork water-shed in 1920: Balch Camp (1 240 ft.); Cliff Camp (6 150 ft.); Helms Creek (called Sand Meadow) (8 020 ft.); and Meadow Brook (8 140 ft.). In 1924 a fifth station was established on Rancheria Creek (6 400 ft.). On the West Fork there are two stations, near Balch Camp (1 310 ft.), and Dinkey Meadows (5 440 ft.). These stations have been maintained in co-operation with the U. S. Geological Survey. Flood run-off relations, which might be called secondary relations in contrast to the primary relations existing between the eleven different water-sheds, were deter-

mined for the main Kings River Station at Sanger and the North Fork Stations. These relations were expressed in second-feet per square mile for varying average elevations of water-shed (Fig. 13) and for drainage areas, in square miles (Fig. 14). As there have been few floods on the North Fork since the stations were established, it was necessary to consider only the corresponding term of years at Sanger. Therefore, as more data are made available the run-off per square mile will be increased, but the relative value

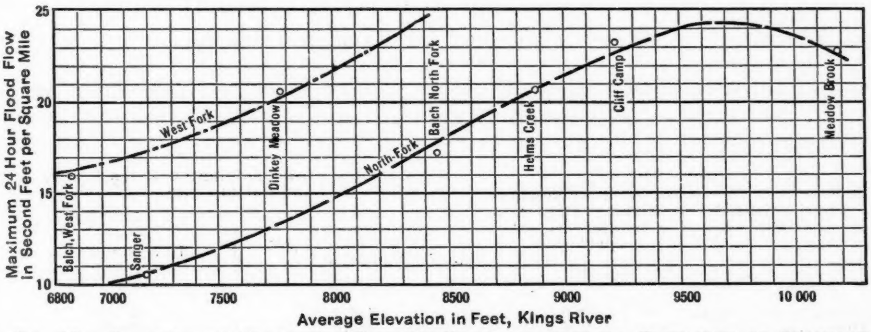

Fig. 13.—Relation of Flood Run-Off (24-Hour) to Average Elevation of Drainage Area of Kings River, California.

from station to station should hold true. Insufficient data were at hand to determine variation factors for the North Fork Stations. As the relative difference in factors which determine run-off, such as slope, exposure to storm, forest and vegetable cover, etc., was small between Sanger and Balch Camp, the difference in variation factors should be slight. However, if the probable flood at an upper station, as Cliff Camp, were being estimated, it would be necessary to take the ratio of variation factors into consideration.

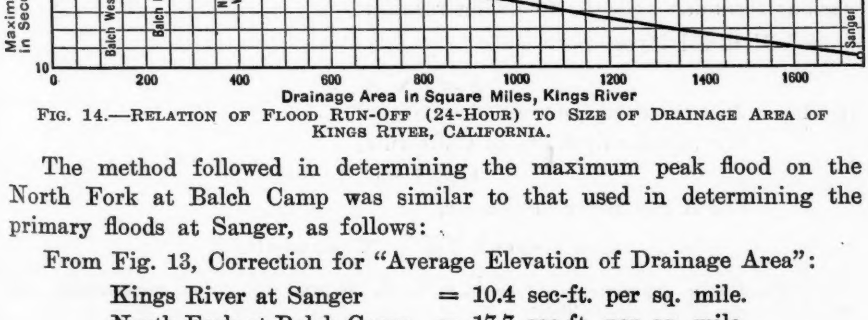

Fig. 14.—Relation of Flood Run-Off (24-Hour) to Size of Drainage Area of Kings River, California.

The method followed in determining the maximum peak flood on the North Fork at Balch Camp was similar to that used in determining the primary floods at Sanger, as follows:

From Fig. 13, Correction for "Average Elevation of Drainage Area":

Kings River at Sanger	= 10.4 sec-ft. per sq. mile.
North Fork at Balch Camp	= 17.7 sec-ft. per sq. mile.
Correction	= 170%

From Fig. 14, Correction for "Drainage Area in Square Miles":

Kings River at Sanger = 10.4 sec-ft. per sq. mile.
North Fork at Balch Camp = 18.7 sec-ft. per sq. mile.
Correction = 180%

The maximum 24-hour flood on the San Joaquin adjusted to suit Kings River conditions produced a peak flood of 36.9 sec-ft. per sq. mile at Sanger. The corresponding peak at Balch Camp would be:

$$36.9 \times 180\% \times 170\% = 112.8 \text{ sec-ft. per sq. mile} = 27\,750 \text{ sec-ft.}$$

The results of the complete study are given in Table 4.

TABLE 4.—DEDUCED FLOOD FLOW FOR NORTH FORK OF KINGS RIVER AT BALCH CAMP, CALIFORNIA.

Flood adjusted from:	Peak flood on Kings River, at Sanger, in second-feet per square mile.	PEAK FLOOD, NORTH FORK OF KINGS RIVER, AT BALCH CAMP.	
		In second-feet per square mile.	In second-feet.
Kern	42.6	130.0	32 000
Tule	42.6	130.0	32 000
Kaweah	31.5	96.5	23 800
Kings	42.2	129.0	31 800
San Joaquin	36.9	112.8	27 750
Fresno	41.4	126.8	31 200
Merced	30.0	91.9	22 700
Tuolumne	32.0	97.8	24 100
Stanislaus	31.1	95.2	23 500
Mokelumne	30.2	92.5	22 800
Cosumnes	33.1	101.2	25 000
Average	35.78	109.5	26 970

If the values given by the adjusted floods of the Kern and Tule Rivers are excluded, the average value of peak flood for the North Fork at Balch Camp is 25 850 sec-ft., or 104.9 sec-ft. per sq. mile, which does not seem excessive. This value has been accepted for designing the waterway area.

COMPARISON OF RESULTS

For comparison and a check, the probable flood for Kings River, at Sanger, and for the North Fork of Kings River, at Balch Camp, was computed from various formulas with the following results:

(*a*) Modified Myers Formula*:

$$Q = C\sqrt{M}$$

in which, M is the drainage area, in square miles.

For interior drainage of California,

$$C = 1\,860$$

For Kings River at Sanger,

$$Q = 1\,860\sqrt{1\,740} = 77\,500 \text{ sec-ft.}$$

* "Flood Flow Characteristics," by C. S. Jarvis, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 985.

For North Fork of Kings River at Balch Camp,

$$Q = 1\ 860 \sqrt{246.4} = 29\ 150 \text{ sec-ft.}$$

(*b*) Fuller Formula:*

$$Q(\text{average}) = C\,A^{0.8}$$

in which, $C = 70$.

For Kings River at Sanger,

$$Q(\text{ave.}) = 27\ 400 \text{ sec-ft.}$$

For North Fork of Kings River at Balch Camp,

$$Q(\text{ave.}) = 5\ 735 \text{ sec-ft.}$$

$$Q(\text{24-hour flood in } T\text{-years}) = C\,A^{0.8}\,(1 + 0.8 \log T)$$

in which, $T = 100$ years.

For Kings River at Sanger,

$$Q(\text{in } T\text{-years}) = 71\ 300 \text{ sec-ft.}$$

For North Fork of Kings River at Balch Camp,

$$Q(\text{in } T\text{-years}) = 14\ 920 \text{ sec-ft.}$$

$$Q(\text{max.}) = Q(1 + 2\,A^{-0.3})$$

in which Q = 24-hour flood in T-years.

For Kings River at Sanger,

$$Q(\text{max.}) = 86\ 500 \text{ sec-ft.}$$

For North Fork of Kings River at Balch Camp,

$$Q(\text{max.}) = 20\ 650 \text{ sec-ft.}$$

(*c*) *Bulletin No. 5,* "Flow in California Streams", Department of Public Works, California:

For Kings River, at Sanger,

$(T = 100 \text{ years}) = 41\,000$ sec-ft. (for a 24-hour flood)

Monthly Distribution of Flood Peaks

Peak floods on California streams of the west slope of the Sierras never occur before December. They are most frequent in January and begin to decrease in frequency and duration in February. These winter storm peaks are caused by the direct precipitation of rain and snow. Beginning with March and ending in June, the flood peaks are produced by the melting of snow (Fig. 15). Winter floods produce a sudden extreme peak of short duration, whereas the summer flood has a long sustained moderately high stage. The Tule River is very flashy and, as is shown by Fig. 15(*a*), in three months of the peak season—December, March, and April—the greatest variation of the peak flood from the 24-hour flood occurred on the Tule. However, the greatest variation, 320%, which was produced by a winter storm in January, took place on the Kings River.

Expectancy of Flood Occurrence

The fact that flood run-off above the mean is more variable than flood flow below the mean has been long established. Rainfall follows the same

* "Flood Flows," by Weston E. Fuller, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.

rule. Flood records for any stations covering a period of years will show a few great floods of high variation from the mean and several floods near or slightly below the average. The length of record on any of the California

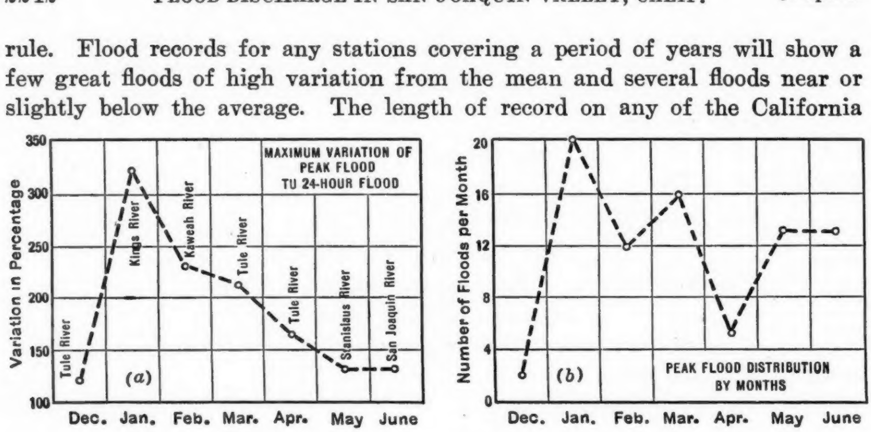

FIG. 15.—FREQUENCY AND INTENSITY OF PEAK FLOODS, ON ELEVEN CENTRAL CALIFORNIA STREAMS.

streams is too short to make an accurate forecast of the frequency of maximum floods. A graphical analysis of the frequency of Kings River floods at Sanger is given in Fig. 16, the data used being shown in Table 5. By

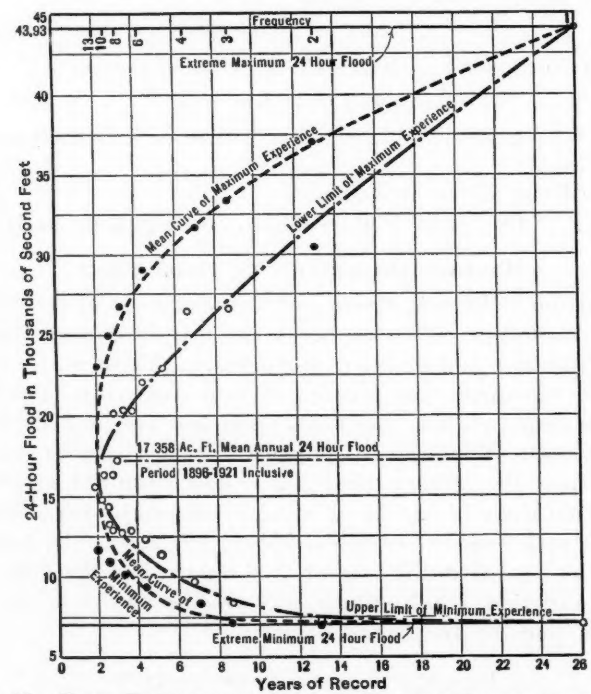

FIG. 16.—FLOOD FREQUENCY ON KINGS RIVER, NEAR SANGER, CALIFORNIA.

prolonging this curve in both directions it is seen that greater and lesser floods may be expected when a greater number of records are available. No

trustworthy estimate is posssible by this method, however, for judgment would indicate that both maximum and minimum curves should become parallel with the base when the most extreme conditions are reached; but what those extreme conditions are, is problematic.

TABLE 5.—EXPERIENCE TABLE OF ANNUAL 24-HOUR FLOOD AT PIEDRA, CALIFORNIA.

Order of magnitude.	FLOODS IN ORDER OF MAGNITUDE, IN ACRE-FEET.		Number of annual floods averaged.	MEAN ANNUAL FLOODS, IN ACRE-FEET.	
	Maximum,	Minimum.		Maximum.	Minimum.
1	43 930	6 920	1	43 930	6 920
2	30 400	7 210	2	37 165	7 065
3	26 600	8 348	3	33 643	7 493
4	26 380	9 795	4	31 827	8 068
6	22 100	12 400	6	28 690	9 312
8	20 300	12 800	8	26 618	10 172
10	17 290	13 200	10	25 043	10 737
13	15 700	15 600	13	22 979	11 659

STORAGE

On a majority of the eleven streams considered, reservoirs of relatively large capacity have been constructed or are contemplated. These reservoirs are in the foothills and their primary use will be for irrigation. Supplemented by storage for power purposes near the head-waters, they will have some favorable effect on the flood flow on the tributary rivers. Being constructed principally for irrigation, a reservoir might be full when a great flood comes; nevertheless, to a certain extent, it would elongate the flood wave and thereby reduce the peak discharge. It would be a great community benefit if a certain proportion of the total capacity could be held for flood protection until the flood menace of a high year was past. This is practiced with a few of the larger U. S. Reclamation Service reservoirs. Although the principal effect of such reservoirs will be to reduce the flood peaks, they will probably have comparatively little effect on the maximum discharge of the extreme flood, which is expected only once or twice in a century.*

FLOOD ESTIMATES

Floods on any water-shed are caused by a storm of a certain type, which varies but little from year to year. If flood records of at least 100 years were available for each stream, it is evident, of course, that the best way to investigate the variations of flood discharge would be to treat each stream by itself. However, on many streams few or no records are at hand. A fair estimate may be had by adjusting the recorded discharges of neighboring streams, similar to the stream in question in geologic, topographic, and meteorologic characteristics. The adjusting factors may not be the same in

* The relation of these foothill irrigation reservoirs to power development is discussed by William Kelly, M. Am. Soc. C. E., in his paper entitled "Co-Ordination of Irrigation and Power," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1429.

all cases, but size of drainage area will be a persistent influence. This comparison or adjustment of existing flood records should be supplemented by the solution of reliable flood formulas, by a study of rainfall records by the time-area-depth method, by reliable evidence of flood deposits, as drift and alluvial bars, and the experiences of observers. Precipitation records, especially if the precipitation is in the form of rain, will serve as a basis of flood prediction, under ideal conditions. Under actual conditions a given rainfall intensity will yield extreme variations of run-off.

Conclusions

In this paper, the writer does not assume to solve the wide problem of flood flows, but merely to describe one method of attack. The future can best be predicted by a study of the past. A rational study of past flood records should form the basis of estimating probable future floods on any stream. The full purpose will be accomplished when a better appreciation of flood danger is held by the Engineering Profession and by the general public. Only after this knowledge is general will the proper measures of flood control be taken.

In addition to the specific acknowledgments made in the paper, the writer has obtained valuable data from the published writings of many engineers, most of them members of this Society. This study was made for the San Joaquin Light and Power Corporation, under the general direction of Mr. J. W. Jourdan. H. K. Fox, M. Am. Soc. C. E., Construction Superintendent, is in charge of the Kings River Development.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

RE-DESIGNING CATAWBA STATION FOR SERVICE ON A LARGE TRANSMISSION SYSTEM*

By W. S. Lee,† M. Am. Soc. C. E.

Synopsis

This paper describes the re-design and re-construction of the first hydro-electric power station of the Duke Power System in the Carolinas, now comprising twelve water-power and six steam-power plants of an aggregate installed capacity of 749 925 kv-a., feeding a transmission system of approximately 3 000 miles of electric circuits.

This plant, located on the Catawba River near Rock Hill, S. C., had an original installation of eight rope-driven generators of 5 600 kw. total capacity. The head of water was 23 ft. Operation of the plant began in 1904, and power was transmitted at 11 000 volts to adjacent territory. After 20 years' operation of the plant, it was decided to raise the dam 47.6 ft., and the reconstructed plant is now operating under a maximum head of 70 ft. The new power-house installation consists of four, vertical-shaft, water-wheel driven generators of 60 000 kw. total capacity. The current is stepped up to 100 000 volts for transmission.

Since the beginning of the Twentieth Century the electric light and power industry has developed rapidly and a modern hydro-electric system now consists of a group of stations of various kinds, which may be described, as follows:

First.—Stations that use the fall and run of the river.

Second.—Stations that use the fall and run of the river plus a small amount of storage, taking care of the night and Sunday flow and perhaps of minor weekly fluctuations of the flow of the river.

Third.—Stations that have a large storage capacity and are used as a valve or outlet to supply the deficiency in power of other stations during low-water seasons.

Note.—Written discussion on this paper will be closed in **March, 1928.**

* Presented at the meeting of the Power Division, Asheville, N. C., April 21, 1927.

† Chf. Engr., Southern Power Co., Charlotte, N. C.

In addition to the hydro-electric stations, the present large power system usually comprises a number of steam-power stations which, preferably, are located at, or in the proximity to, industrial centers so as to reduce the losses in transmitting the power to the points of delivery. They have various efficiency ratings, depending on the time they are intended to be operated in conjunction with the hydro-electric plants, and they may be divided into the following classes:

1.—Plants designed to operate intermittently and during low-water periods; termed "stand-by plants".

2.—Plants designed to operate twelve months of the year and capable of supplying a constant amount of power; termed "base plants".

Finally, by interconnecting the trunk lines of the transmission systems of large independent power companies operating in adjoining districts, energy can be transferred from one district to another in a so-called super-power zone, the principal object being to insure a reliability of service and to make possible a more effective utilization of the water power from streams having different characteristics of flow.

The Duke Power System, in the Piedmont Section of the Carolinas, is now operating twelve water-power plants and six steam-power plants which may be grouped as follows:

	Installed capacity, in kilovolt-amperes.
A.—Run of River Hydro-Electric Stations:	
Greatfalls	30 000
Dearborn	56 250
Total	86 250
B.—Hydro-Electric Stations with Storage Ponds:	
Lookout Shoals	22 800
Rocky Creek	30 000
Cedar Creek	56 250
99 Islands	22 500
Total	131 550
C.—Hydro-Electric Stations with Large Storage Reservoirs:	
Bridgewater	25 000
Rhodhiss	31 875
Mountain Island	75 000
Catawba	75 000
Fishing Creek	37 500
Wateree	70 000
Total	314 375
D.—Stand-by Steam Stations:	
Greenville	8 000
Greensboro	8 000
Total	16 000

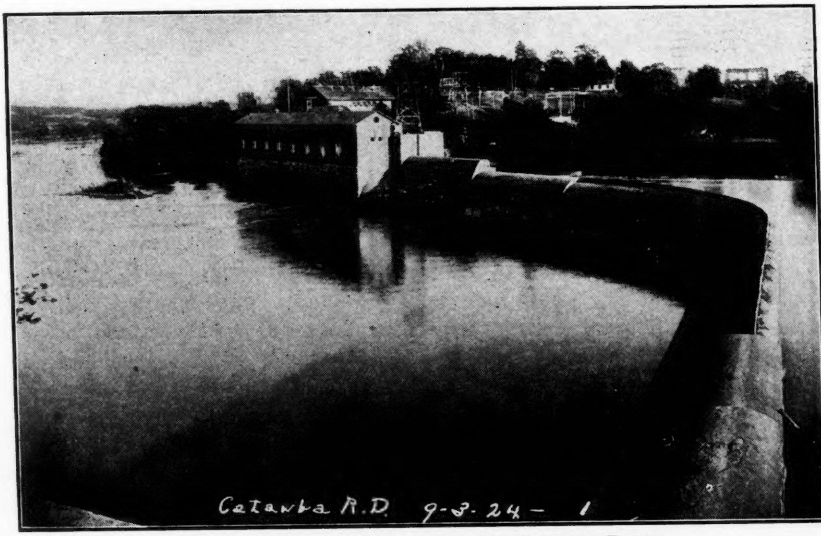

FIG. 1.—GENERAL VIEW OF THE OLD CATAWBA PLANT.

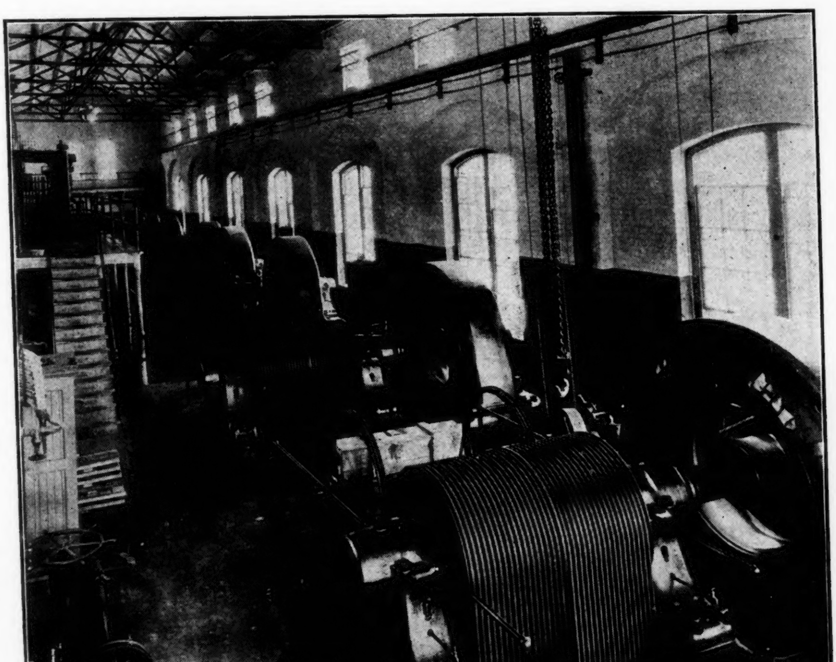

FIG. 2.—INTERIOR VIEW OF OLD CATAWBA STATION.

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	Installed capacity, in kilovolt-amperes.
E.—Base Steam Stations:	
Tiger	37 500
Mount Holly	45 500
Eno	31 250
Buck	87 500
Total	201 750

These power stations, of an aggregate installed capacity of 749 925 kv-a., feed into a transmission system consisting of approximately 3 000 miles of circuits, of which more than 50% are 100 000-volt, steel-tower lines. These power lines serve a territory approximately 350 miles long and 150 miles wide, and the trunk lines are interconnected with the transmission systems of three independent companies all of which operate in the Southeastern Power Zone comprising the States of North Carolina, South Carolina, Georgia, Alabama, and Tennessee.

In 1904, when the Catawba Power Company, now forming part of the Duke Power System, began operation of the 10 000 h. p. development on the Catawba River at India Hook Shoals, 6½ miles north of Rock Hill, S. C., three, 11 000-volt, pole lines of 28 miles total length, carried the electric current to the Towns of Rock Hill and Fort Mill, S. C., and Charlotte, N. C., which latter point was 18 miles distant from the power plant.

Old Catawba Station

The first plant of the Duke Power System was called the Catawba Station (Fig. 1), and contained eight, horizontal shaft, rope-driven generators, four of 750 kw. each, and four of 900 kw. each, making a total rated capacity of 6 600 kw. (Fig. 2). The speed of the horizontal shaft, triple turbines was 110 rev. per min., and that of the three-phase, 60-cycle, 11 500-volt generators, 300 rev. per min. The normal head of water at the station was 23 ft. and the guaranteed efficiencies were, as follows:

	Efficiency, percentage.
Turbines	80
Generators	94
Rope drive	94
Over-all efficiency	70.7

The highest flood on record at that time was 27.1 ft. above normal tail-water level, and the generator floor was 1 ft. higher, the station arrangement being such that good light and cool air could be obtained through windows on both sides of the power house.

The impounding structure of the plant consisted of a curved spillway 585 ft. long of standard ogee type, built of bulk concrete masonry, with an average height of 34 ft. above the bed-rock foundation. An earth dam, 400 ft. long, with slopes riprapped on both sides, was rebuilt and raised after the July, 1916, flood, when the river discharge exceeded the 1901 flood by 100 per cent.

The drainage area of the Catawba River at the station is 3 085 sq. miles, the pond area was 1 800 acres, and the available storage capacity, 202 000 000 cu. ft. Before the large storage reservoirs were built on the Upper Catawba River, the stream carried large volumes of silt in suspension during flood stages, a considerable amount being deposited in the Catawba pond, so that after twenty years' operation of the plant barely the night low-water flow of the river could be stored when the plant was not operated 24 hours.

Following the July, 1916, flood, the Duke Power Company built three large storage reservoirs on the Catawba River above the station for the purpose of impounding the high waters of the Upper Catawba River for use during periods of low water on the Lower Catawba. This resulted in a large increase of uniform flow at the Catawba Station and the question arose whether additional machinery should be installed and the old plant overhauled; whether the dam should be raised so that the water would back to the tail-race of the Company's Mountain Island Station, 25 miles farther up stream, thus creating another large storage reservoir; or whether to build an entirely new power house and scrap the old power plant representing a value of approximately $1 000 000.

A topographic survey of the entire storage basin was then made, and it was found that for a development of 70-ft. head, which calls for a raising of the old spillway 47.6 ft. above the top of the flash-boards, the superficial area of the full reservoir is 12 455 acres and the available storage capacity 10 300 000 000 cu. ft. considering a 30-ft. draw-down. In view of the fact that this storage was available not only for the new Catawba Station, but also for the existing stations of the Duke Power Company, on the Lower Catawba River, having a combined mean head of 250 ft., authority was given, in August, 1924, to raise the old dam and to build a new power station.

Designing Features of New Plant

As shown in Fig. 3, the new station is on the left side of the river, looking down stream, and the lower part of the south end of the substructure is built against the retaining wall for the old earth fill. The developed length of the new spillway is 802.93 ft. measured between centers of flood-gate piers. Referring to Fig. 4, the spillway contains eleven steel flood-gates, 45 ft. wide by 30 ft. high, and one smaller gate, 19 ft. wide and 25 ft. high. The curved section of the spillway, 168.52 ft. long, contains no flood-gates and the top is at Elevation 570.0, corresponding to the top of the flood-gates when in closed position. The space occupied by the masonry supporting gates, Nos. 9, 10, and 11, and the small gate, contains the substructure of the old power house.

The raised spillway follows the same alignment as the old spillway and the new masonry is built on top, against the down-stream face of the old masonry as indicated on Fig. 5. Three, 12 by 12-in. drains running along the rollway of the old structure are formed in the new masonry and are provided with 12 by 12-in. side outlets, at 55-ft. intervals, by which any seepage water from the pond will be discharged down stream. A down-stream view of the new spillway under construction, with the new power house on the right-hand side and the partly demolished old power house on the left-hand side, is shown on Fig. 6.

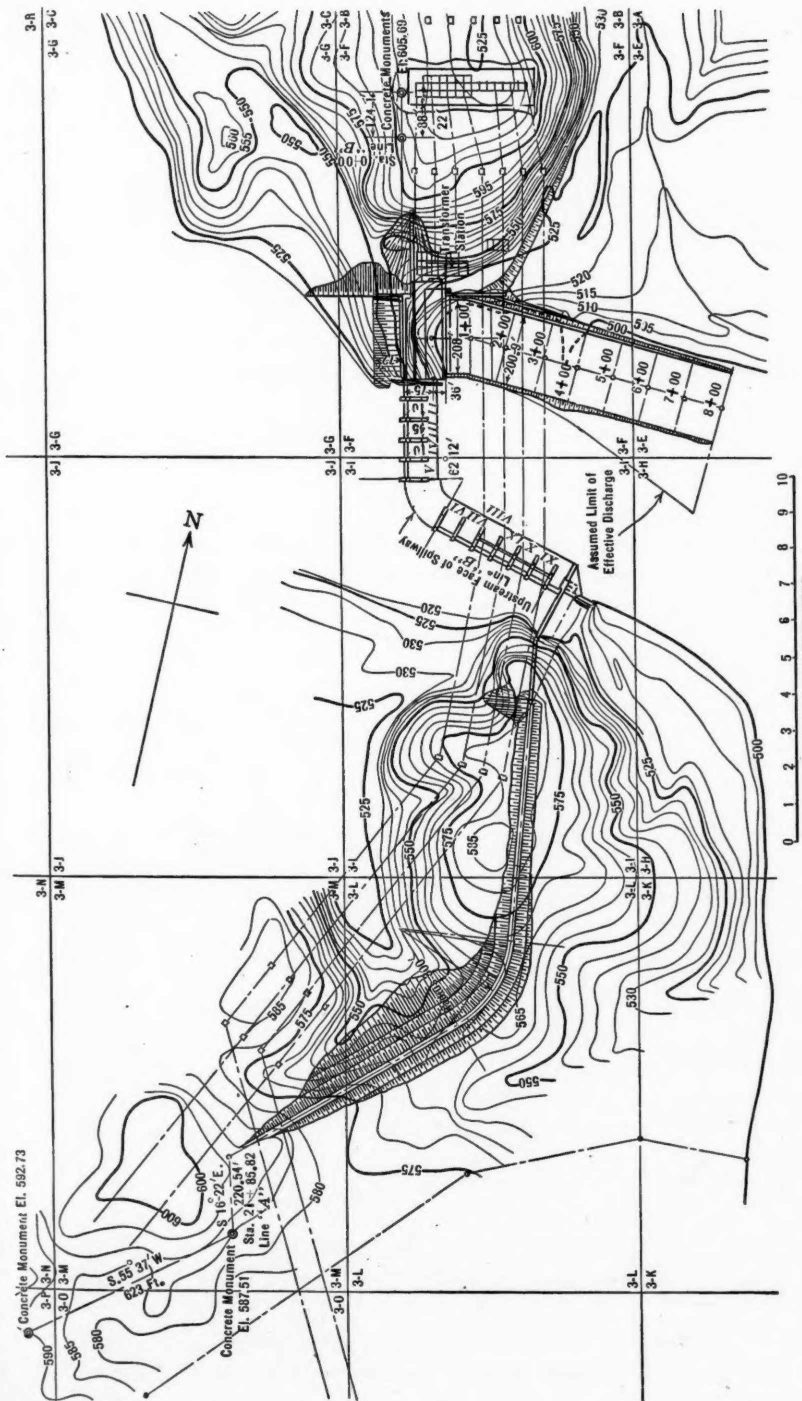
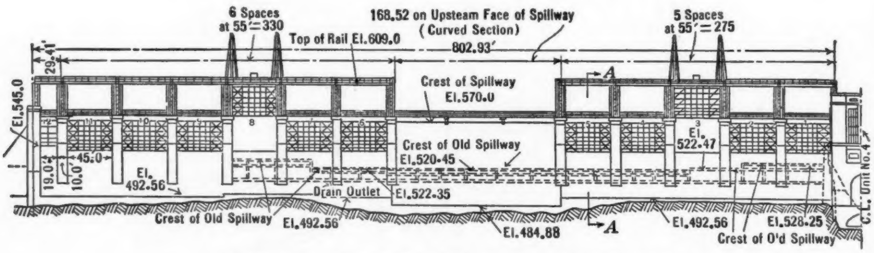


FIG. 3.—GENERAL PLAN OF NEW CATAWBA STATION.

The power house substructure forming the down-stream extension of the bulkhead is built of concrete masonry strengthened by reinforcing steel around the four wheel chambers. The draft-tubes are formed in the concrete and only the upper part is lined with plate steel so as to prevent wearing of



DEVELOPED DOWNSTREAM ELEVATION OF SPILLWAY

FIG. 4.—NEW CATAWBA STATION. ELEVATION OF SPILLWAY.

the concrete surfaces due to the high velocity of the water discharging from the turbines at large gate-opening. The generator floor is at Elevation 545.0, which is 21.5 ft. higher than the generator floor of the old plant. Fig. 7 is a general view of the new station.

FIG. 5.—NEW CATAWBA STATION. CROSS-SECTION *A-A* OF SPILLWAY.

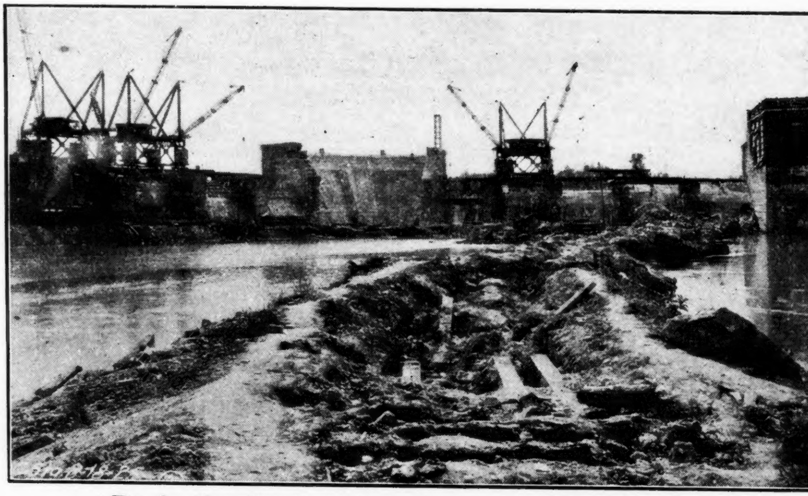


FIG. 6.—NEW SPILLWAY UNDER CONSTRUCTION, CATAWBA PLANT.

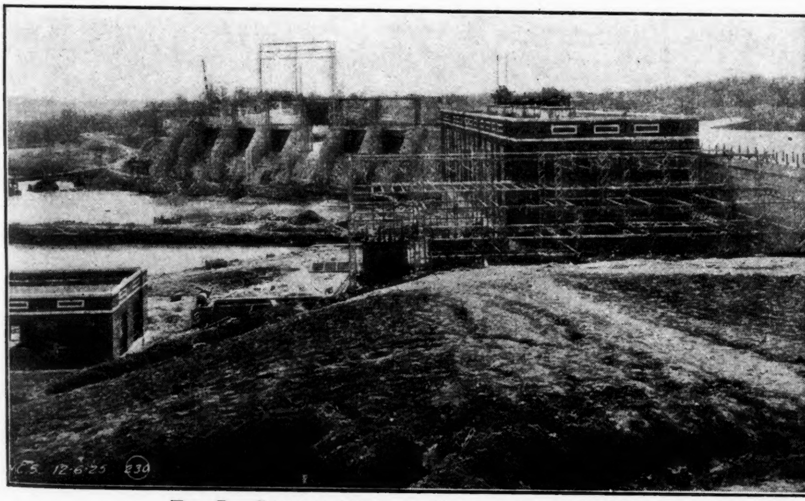


FIG. 7.—GENERAL VIEW OF NEW CATAWBA STATION.

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The rack structure at the up-stream face of the power house bulkhead is built of reinforced concrete and structural steel, the spacing of the rack bars being $4\frac{1}{2}$ in., center to center. The intake to each turbine flume consists of two 18 by 25-ft. openings formed in the concrete masonry. Each opening contains a rectangular gate of the butterfly-valve type, having in the center a cast

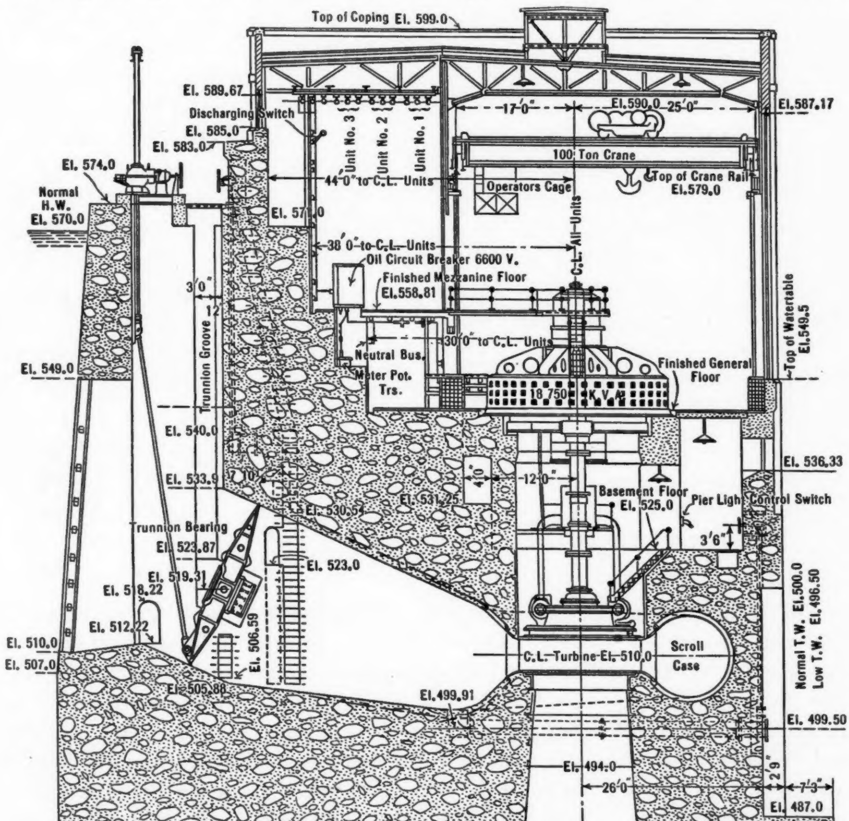

FIG. 8.—CROSS-SECTION, POWER HOUSE, NEW CATAWBA STATION.

steel girder, 3 ft. deep and 2 ft. 8 in. wide, with bronze-bushed trunnions, 28 in. in outside diameter and 20 in. long, at each end. The wings of cast iron are firmly attached to the girder by machine bolts and shrink rods. A built-up cast-iron wall-frame securely anchored to the masonry is provided with machine-finished surfaces at its lower half at which point the surfaces of the gate and wall-frame come in direct contact. The upper half of the wall-frame has machine-finished beveled faces conforming to similar surfaces of the gate. The slot between the upper half of the gate and the wall-frame is sealed by a

set of brass flaps attached to the up-stream side of the frame. Each gate is operated by an individual, completely enclosed, electrically operated hoist on top of the power house bulkhead as shown in Fig. 8. A gate-position indicator is provided on top of the hoist. Normally, the head-gate motor will be operated by means of a push-button control system on top of the bulkhead and in case of emergency the push-button control on the power house benchboard will be used.

The superstructure of the power house is built of brick encasing the structural steel frame that supports the roof trusses and the steel runways for the 100-ton electric crane in the generator room. The outside of the building is of red tapestry brick with terra cotta trimmings, and the interior of the station is covered with plaster and finished with a coat of paint. The generator floor and the inside face of the concrete water-table for a height of 4.5 ft., are covered with glazed tile. The large window sashes are built of steel and the roof consists of reinforced gypsum slabs resting on steel purlins, the top of the slabs having a finish of tar and gravel.

The power house contains four vertical-shaft, single-runner, Francis type turbines operating at 100 rev. per min. Between 68.5 and 40-ft. head, the rating of each turbine is, as follows:

Head, in feet	68.5	65	60	50	40
Maximum horse power..	22 500	21 000	19 000	14 800	10 600
Best gate-opening.......	0.85	0.86	0.87	0.88	0.96
Efficiency, in percentage, at best gate-opening...	89.5	90	90	87.5	84

An individual governor system, operating under an oil pressure up to 200 lb. per sq. in., is provided for each turbine, and the piping is arranged so that the governors may be operated separately or as an interconnected system. The governor fly balls are directly connected to the turbine shaft below the generator rotor. An emergency-stop device is installed for each turbine, and is designed to shut the turbine gates and stop the unit from the station switchboard. Fig. 9 is an interior view of the new Catawba Station.

The three-phase, 6 600-volt, 60-cycle, vertical shaft generators are directly connected to the respective turbines. Each unit is rated at 18 750 kv-a, or 15 000 kw., at 80% power factor. Individual exciters are mounted on top of the generators and a unit oiling system is provided for each machine. The guaranteed efficiencies when operating at 80% power factor are: Full load, 96.3%; three-quarters load, 96.0%; and one-half load, 95.0 per cent.

The bench-board is on the mezzanine floor at the north end of the power house. Through four water-cooled, outdoor-type, transformers of the same capacity as the generators the current is stepped up to 100 000 volts and then fed into the transmission system of the Duke Power Company. A view of the outdoor transformer station nearing completion is shown in Fig. 7.

The tail-race is 800 ft. long, and if the four units are operated at seven-eighths gate-opening and under 65-ft. head, the discharge is 12 000 cu. ft. per sec. Under these conditions the mean velocity of the water in the smallest tail-race section is 4.0 ft. per sec.

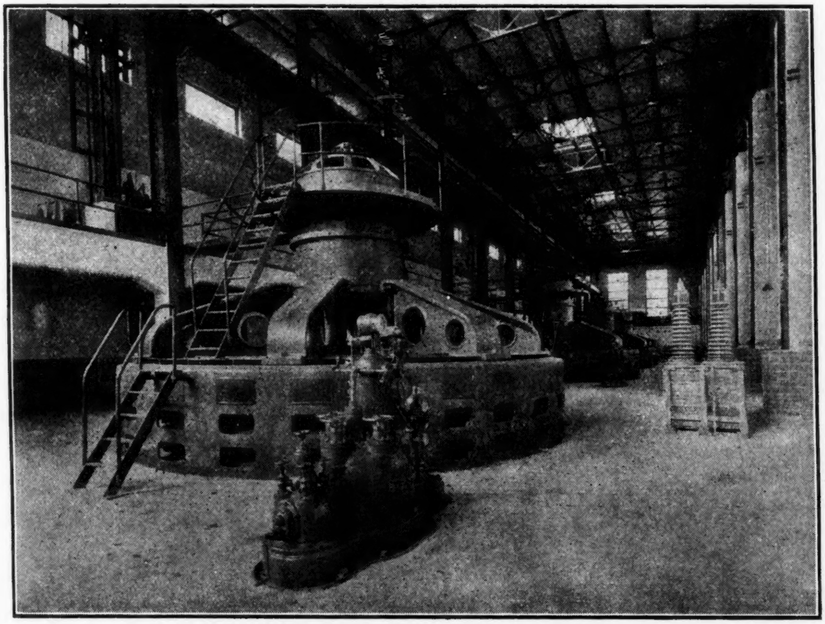

Fig. 9.—Interior View of New Catawba Station.

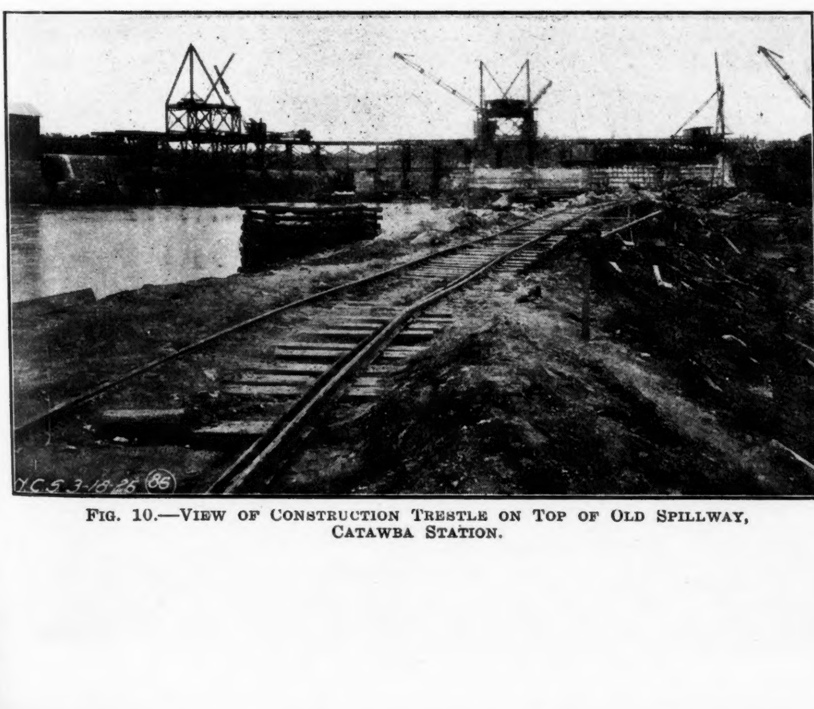

Fig. 10.—View of Construction Trestle on Top of Old Spillway, Catawba Station.

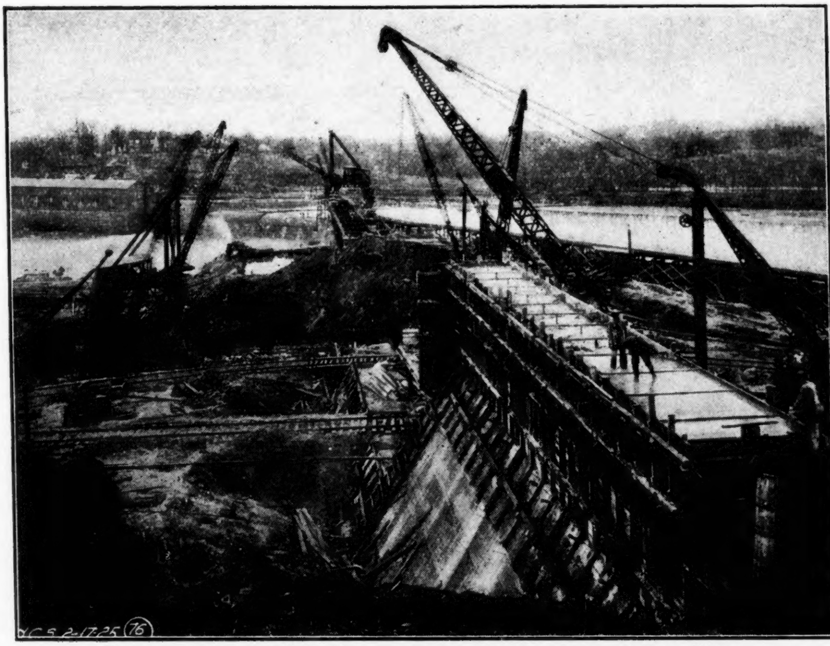

FIG. 11.—SITE OF NEW POWER HOUSE, SHOWING EXCAVATION WORK BELOW OLD EARTH FILL.

FIG. 12.—CRUSHER AND MIXER PLANT, CATAWBA STATION.

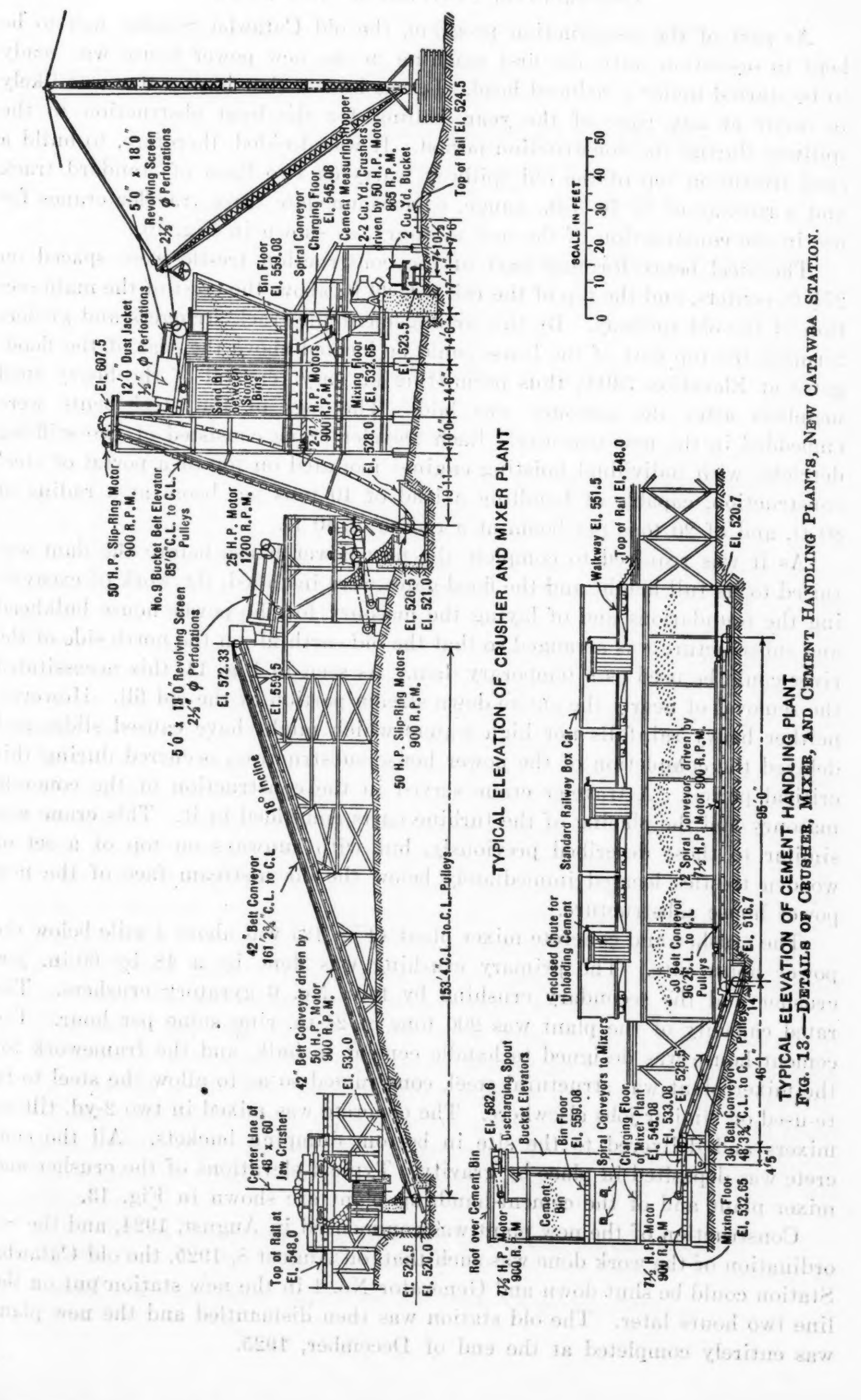

FIG. 13.—DETAILS OF CRUSHER, MIXER, AND CEMENT HANDLING PLANTS, NEW CATAWBA STATION.

Construction Features of New Plant

As part of the construction program, the old Catawba Station was to be kept in operation until the first machine in the new power house was ready to be started under a reduced head. It was known that high water was likely to occur at any time of the year, calling for the least obstruction to the spillway during the construction period. It was decided, therefore, to build a steel trestle on top of the old spillway, carrying two lines of standard track and a runway of 32 ft. 3-in. gauge, to accommodate three traveler cranes for use in the construction of the new masonry, as shown in Fig. 10.

The steel bents forming part of the construction trestle were spaced on $27\frac{1}{2}$-ft. centers, and the top of the rail was 26 ft. above the crest of the main section of the old spillway. By this arrangement the track stringers and girders forming the top part of the bents could be located above the seat of the flood-gates at Elevation 540.0, thus permitting an easy removal of the heavy steel members after the masonry was laid. The legs of the steel bents were embedded in the new masonry. Each traveler crane consisted of two stiff-leg derricks, with individual hoisting engines mounted on top of a portal of steel construction, capable of handling a load of 10 tons per boom at a radius of 80 ft. and of 20 tons per boom at a radius of 40 ft.

As it was required to complete the new power house before the dam was raised to its full height and the flood-gates were installed, the work of excavating the foundations and of laying the masonry for the power house bulkhead and substructure was arranged so that the old earth fill on the north side of the river could be used as a temporary dam. As seen in Fig. 11, this necessitated the removal of nearly the entire down-stream portion of the old fill. However, neither heavy rainfalls nor high waters which might have caused slides and delayed the completion of the power house substructure, occurred during this critical period. A traveler crane served in the construction of the concrete masonry and the placing of the turbine parts embedded in it. This crane was similar to those described previously, but with runways on top of a set of wooden trestles located immediately below the down-stream face of the new power house substructure.

The crusher and concrete mixer plant (Fig. 12) was about $\frac{1}{4}$ mile below the power house site. The primary crushing was done by a 48 by 60-in. jaw crusher and the secondary crushing by four No. 6 gyratory crushers. The rated capacity of the plant was 200 tons of $2\frac{1}{2}$-in. ring stone per hour. The cement house was designed to handle cement in bulk, and the framework for the mixer plant was structural steel, constructed so as to allow the steel to be re-used on similar jobs elsewhere. The concrete was mixed in two 2-yd. tilting mixers and delivered to the site in bottom dumping buckets. All the concrete was deposited in place by gravity. Typical elevations of the crusher and mixer plant and of the cement-handling plant are shown in Fig. 13.

Construction of the new plant was commenced in August, 1924, and the co-ordination of the work done was such that, on August 8, 1925, the old Catawba Station could be shut down and Generator No. 1 in the new station put on the line two hours later. The old station was then dismantled and the new plant was entirely completed at the end of December, 1925.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE*

By Charles Terzaghi,† M. Am. Soc. C. E.

Synopsis

The paper reviews the present state of the science of foundations, its principal shortcomings, and the possibilities for its improvement.

The principal shortcomings were found to be: First, the practice of selecting the admissible soil pressure regardless of the area covered by the individual foundations and irrespective of the maximum differential settlement that the superstructure can stand without injury; second, the practice of computing the bearing capacity of the piles by the *Engineering News* formula regardless of the character of the soil; and, third, the practice of considering the bearing power of the individual piles as a sufficient guaranty that the bearing capacity of the entire foundation will be adequate.

In his discussion of these topics the writer tries to explain the reasons for the inconsistencies that are often experienced in the attempts to interpret the results of loading tests for designing purposes. Concerning the pile-driving formulas the writer presents physical arguments why, for certain soils, no pile-driving formula can possibly furnish reliable information concerning the bearing capacity of the tested pile. For those soils where the pile-driving formulas can be used to advantage, the *Engineering News* formula is found to furnish values which are by far too small, provided a drop-hammer was used and the penetration per blow amounted to less than ½ in.

Progress in the field of foundation engineering is handicapped essentially by the absence of reliable information concerning previous construction experience. Again, the lack of reliable information is due to inadequate description of the soils and to an interpretation of the observed facts which, in many cases, is inconsistent with the laws of physics and mechanics. The first step toward improving conditions should consist in establishing an adequate soil classification.

Note.—Written discussion on this paper will be closed in March, 1928.

* Presented at the meeting of the Structural Division, New York, N. Y., January 20, 1927.

† Assoc. Prof., Foundation Eng., Mass. Inst. Tech. Cambridge, Mass.

Thus far, related efforts have failed, because they represented attempts to classify the soils according to properties that have no bearing at all, or no well-defined bearing, on the behavior of the soil under load.

The writer presents a list of those soil properties that determine the character, the amount, and the speed of the settlements. He shows how the knowledge of these properties can be utilized for establishing a soil classification suited for its purpose and briefly discusses the difficulties associated with such an attempt.

The problems of foundation engineering are as inseparably associated with the problems of structural engineering, as is the shadow with the light, because one cannot conceive of a structure without a foundation. Therefore, they are among the most vital.

Effect of Type of Building on Admissible Settlement

Suppose it is desired to construct a reinforced concrete building on a lot selected and bought by the future owner. After inspecting the ground in the light of previous experience, it is decided to dispense with piles and put the building on individual spread footings. The mental operation leading to this decision is experienced almost daily, and is familiar to every engineer. This apparently simple decision may be analyzed by common sense. By spreading the weight of the building over the base of sufficiently large footings, the pressure can be reduced to a certain value, q, per unit of area. Under the influence of the pressure, q, the footings will settle through a distance, S. Perfectly uniform settlement will not hurt the building. However, because of the variations in the distribution of the live load, the pressure may vary between q and $0.8\,q$ (to name an arbitrary figure) which corresponds to a difference in settlement between S and $0.8\,S$. In addition, it may be that the compressibility of the ground varies to such an extent, that the settlement produced by a load, q, may range between S and $0.5\,S$ (which, again, is an arbitrarily selected figure, the actual value depending on circumstances). Hence, a difference, amounting to as much as $0.6\,S\ (= H)$, may be expected between the settlement of the individual footings. In the formula,

$$H = 0.6\,S = n\,S \qquad (1)$$

factorial n represents the numerical effect of both the variations in live load and the variations in the compressibility of the soil. The value, H, will rule all further considerations. If the building is statically determinate a value of several inches for H would be permissible. On the other hand, if it is statically indeterminate, the allowable intensity of the secondary stresses must be selected first. Consider, for example, that an increase of 15% in certain maximum bending moments would be admissible. Then, the maximum value, H, for the difference in settlements is at once determined. It may range between 0.1 in. and 1.0 or 2.0 in., according to the character of the building, the distance between the columns, and the cross-section of the beams. Hence, the decision to admit an increase of 15% in the dangerous stresses, would require selecting the soil pressure, q, anywhere between limits as far apart

as 1 and 10 or 20, according to the character of the building. Selecting the value, q, on the other hand, irrespective of the structural characteristics of the building, would mean either ultra-conservative wastefulness or danger, according to the circumstances. Since this is done in most of the cases dealt with in actual practice, one cannot help feeling the necessity of a thorough reform. Every foundation design ought to be started by roughly computing the maximum admissible value of H.

Relation Between Settlement, Size of Loaded Area, and Depth of Foundation

The second inconsistency that confronts the foundation engineer daily, concerns the relation between the settlement, S, and the diameter of the loaded area. In selecting the admissible unit soil pressure, should the width and the length of the loaded area be taken into consideration? Consider, as an example, that some one makes a bending test on a 10-in. by 10-ft. I-beam, freely supported at both ends, and finds that the beam breaks under the influence of a concentrated load of P-tons. He publishes the results of his test, and as a consequence, in future practice, the designers would consider a load of $\frac{1}{4} P$ as a safe load for the beam, irrespective of the span and mode of support. In the light of present knowledge, such procedure would be considered simply absurd. However, in the field of foundation engineering it corresponds precisely to what is actually done. The safety of the building depends on H ($= 0.6\ S$) not exceeding a definite value. The value of S depends as essentially on the diameter of the loaded area and on the depth of the foundation as the load causing the rupture of the I-beam depends on the span. Nevertheless, all tables of admissible soil pressures, given in textbooks and building codes, contain the values without any indication as to what areas and to what depth of foundation they apply. They can well be compared with tables containing the loads under which I-beams of different cross-sections broke down, without mentioning the distance between supports.

The rather indifferent attitude of practising foundation engineers toward the important influence of these two items—the dimensions of the loaded area and the depth of foundation—is essentially due to the apparent inconsistencies of related experiences which seem to contradict each other, to such an extent that an attempt to digest them appears to be hopeless. However, by analyzing them and correlating them with present theoretical knowledge, one finds that they already represent a fund of most useful information, and that the apparent contradictions do not exist. The following facts may serve to illustrate these statements.

In 1916, the Foundation Committee of the Austrian Society of Engineers and Architects, in Vienna, made a series of loading tests for the purpose of investigating the influence of the shape and size of the loaded area on the amount of settlement in a typical cohesive Vienna soil (a variety of loess).* The tests were performed on the bottom of a trench, 10 ft. deep and 5.75 ft.

* According to a manuscript, "Der Wiener Löss und seine zulässige Belastung," by Dr. Fritz Emperger, received in May, 1926. According to Dr. Emperger's letter of March 16, 1926, an extract of this paper has been published in *Die Bautechnik*, Berlin, *Le Genie Civil*, Paris, and *Gewappend Beton*, Amsterdam.

wide, protected by a roof. The loaded soil consisted, according to a statement issued by the investigators, of 27% of sand with grains larger than 0.2 mm., 31% of sand with grains less than 0.2 mm., and 42% of very fine-grained constituents. The water content was said to range between 11.2 and 13.2 per cent. When moulded into cubes and dried, the soil developed a cube strength of about 35 tons per sq. ft. The tests included load settlement, time settlement, and rebound observations on square and on round plates with areas of 0.053, 0.110, 0.672, 2.700, and 6.100 sq. ft. Fig. 1 shows some of the results obtained. They include the settlements of plates with different diameters under loads of 2 and of 4 tons per sq. ft., and the rebound of the soil produced by removing a load of 10 tons per sq. ft. (Computed from the rebound, corresponding to the removal of smaller loads.) Fig. 1 plainly demonstrates that the settlement produced by a given load per unit of area increases almost in direct proportion with the diameter of the loaded area, or, in algebraic terms,

$$S = c \times d \times q \text{ (more or less)} \dots\dots\dots (2)$$

wherein c is a constant of the soil. The shape of the upper set of curves is obviously due to the fact that the load-settlement curve for each one of the individual plates is, in itself, more or less parabolic.

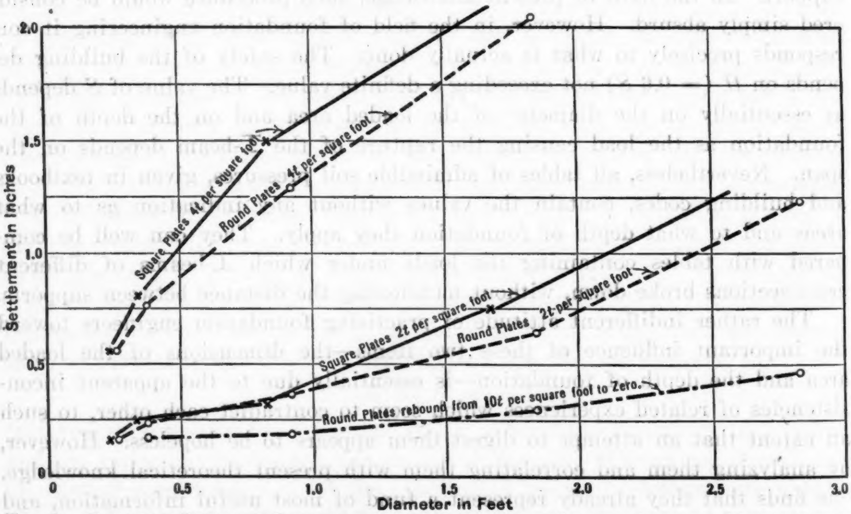

FIG. 1.—RELATION BETWEEN SETTLEMENT AND DIAMETER OF LOADED AREA FOR VIENNA LOESS AT EQUAL UNIT LOAD ACCORDING TO DR. FRITZ EMPERGER.

A few years ago, A. T. Goldbeck, Assoc. M. Am. Soc. C. E., carried on some research work for the U. S. Bureau of Public Roads, on artificial mixtures of sand and clay for the purpose of investigating the relation between the area of a bearing block and the settlement produced by a given unit load.* The area of the bearing blocks ranged from a few square inches to 9 sq. ft. The results of the tests made on plastic mixtures were of the type shown in

* "Researches on the Structural Design of Highways by the United States Bureau of Public Roads," *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 264.

Fig. 2. They indicate, as do the Austrian tests, that for cohesive soils the penetration produced by a given unit load increases in direct proportion with the diameter of the loaded area.

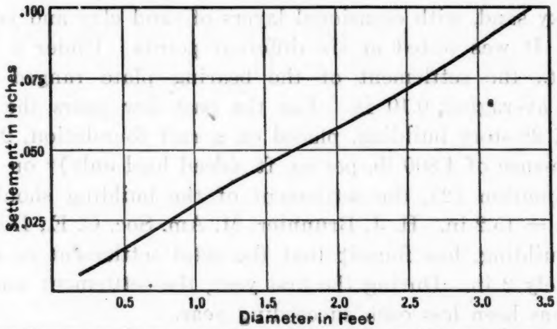

Fig. 2.—Relation Between Settlement and Diameter of Loaded Area at Equal Unit Load, According to A. T. Goldbeck, Assoc. M. Am. Soc. C. E.

In 1923, Mr. A. B. Bijls* published the results of similar tests performed on a sand deposit. These tests, however, cannot be taken into consideration, because the published data are confined to the results of rebound observations.

These experimental results lead to the conclusion that the settlements produced by a given load per unit of area increases approximately in direct proportion with the diameter of the loaded area (Equation (2)), provided the load, q, is not carried beyond the straight line section of the load-settlement diagram.

This agrees with a considerable number of actual observations. Yet there seem to be quite striking exceptions to this rule. A loading test was made by the Chicago Union Terminal Company, in Chicago, Ill., on two full-sized caissons, resting on the surface of a layer of hardpan, approximately 60 ft. below the surface of the ground.† The diameters of the bearing area were 4.3 ft. and 8.5 ft., respectively, and the settlements produced by a load of 10 tons per sq. ft. amounted to ⅛ in. and ⅝ in., respectively. Hence, although the larger diameter was equal to only twice the smaller, the settlement of the larger pier was five times greater than the settlement of the smaller one. According to Equation (2), it should not have been more than twice as large. This fact seems to indicate that the formula represents an optimistic statement.

On the other hand, experience shows that, under certain circumstances, the settlement of the larger area may be very much smaller than the value obtained by the formula. Numerous loading tests made on the plane surface of a natural bed of well-compacted sand, have shown that the settlement of a single bearing plate, with an area of 1 sq. ft., loaded with 3 tons, is never less than about ⅛ in. Hence, according to Equation (2), a foundation 20 ft. square, resting on a firm sand deposit, should settle 2½ in. under a load of 3 tons per sq. ft. Every engineer who has had experience with foundations

* *Le Génie Civil,* May 26, 1923, p. 490. Abstract of a report by Mr. M. Wolterbeck to the Ministère des Travaux Publics des Pays Bas, 1922.

† *Journal,* Western Soc. of Engrs., February, 1924.

on firm sand can testify that the settlement will be very much less, certainly not exceeding ½ in. In 1922, the Society's Special Committee on Bearing Value of Soils for Foundations, etc., published the results of loading tests performed on a building lot in San Francisco, Calif.* A stratum of soft, clean, or sticky sand, with occasional layers of sand clay and yellow clay was encountered. It was tested at six different points. Under a load of 4 800 lb. per sq. ft., the settlement of the bearing plate ranged between 0.04 and 0.17 in., averaging 0.10 in. For the past few years the lot has been occupied by a 22-story building, placed on a raft foundation, 218 by 252 ft., exerting a pressure of 4 800 lb. per sq. ft. (dead load only)† on the soil. According to Equation (2), the settlement of the building should amount to 152×0.10 in. = 15.2 in. H. J. Brunnier, M. Am. Soc. C. E., Designing Engineer of the building, has found‡ that the total settlement to date amounts to approximately 2 in. During the first year, the settlement was quite rapid, but the rate has been less each succeeding year.

Reviewing the facts, the following statement is clear. First, two independent sets of experiments have been quoted concerning the relation between the size of the loaded area and the settlement. Both sets have led to practically the same result, expressed by the simple Equation (2). Then, the results of these experiments have been compared with those of two observations made on a large scale. In one of these cases (Chicago Union Terminal tests), the settlement of the larger area was 2.5 times more than the computed value; while in the other case (San Francisco office building), the actual settlement was less than one-tenth of what should have been expected. Suppose a foundation company sincerely tries to obtain some reliable basis for drawing conclusions from the results of its loading tests. The engineers of the firm make their observations year by year, accumulate the data in their files, and, finally, when they try to correlate the data, they face grotesque contradictions of the type quoted. They cannot be blamed if they give it up in disgust, loading tests, observations, and all.

However, if they would examine the data in the light of applied mechanics, they would first realize the following facts, which the writer published in 1925:§

(*a*) The relation between the diameter of the loaded area and the settlement produced by a given unit load depends essentially on the cohesion (actual shearing strength) of the soil. For soils with great cohesion the settlement produced by a given unit load increases in direct proportion with the diameter of the loaded area. On the other hand, for perfectly cohesionless soils, the size of the area has very little effect.

(*b*) With increasing depth of foundation, the settlement produced by a given unit load decreases. However, the ratio between the settlement S_0 for a foundation depth of 0, and the corresponding settlement S for a depth, t, does

* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 529, Plate VII.

† "Continuous Mat Foundation for 22-Story Building," *Engineering News-Record*, July, 1922, pp. 73 *et seq.*

‡ By letter of December 4, 1925, to Maj. W. A. Danielson.

§ "Erdbaumechanik," by Charles Terzaghi, M. Am. Soc. C. E., Franz Deuticke, Wien, 1925.

not depend on the value of t alone, but on the ratio, $\frac{t}{d}$, between the depth of foundation and the diameter of the loaded area. Thus, if a foundation 5 ft. deep was found to reduce the settlement of a footing, 5 ft. square, by 50% of S_0, the effect of a footing 10 ft. square, and 5 ft. deep, in reducing settlement, will be very much less. In order that the potential settlement S_0 of a footing 10 ft. square, and 0 ft. deep, may be reduced 50%, the depth would have to be increased to 10 ft., thus making the ratio, $\frac{t}{d}$, for both cases equal.

(*c*) The effect of the ratio, $\frac{t}{d}$, on the settlement is less, the greater the cohesion. For perfectly cohesionless materials, a ratio of $\frac{t}{d} = 1$ (depth of foundation = diameter of loaded area) almost triples the bearing capacity and reduces the settlements to one-third of what they would be if the footing rested on the surface of the ground.

Knowing nothing more about soil mechanics than these three simple rules, the foundation engineer would at once discover that there are no contradictions in the data hereinbefore quoted. The experiments of the Austrian Committee and of Mr. Goldbeck were made on very cohesive soils. Hence, in strict agreement with theory, they showed that the settlement increases in direct proportion with the diameter of the loaded area. The Chicago tests were made on a cohesive soil, but the ratio between the depth of foundation and the diameter of the loaded area was twice as great for the larger area as for the small one. That fully accounts for the difference between the computed and the actual value of the settlement of the larger area. On the other hand, the soil of San Francisco belongs to the class of very slightly cohesive soils and, according to Rule (*a*), which is supported by experience, the size of the area should have comparatively little effect on the settlement.

Thus, if it combines actual observation with proper soil investigation and with a thorough consideration of the mechanical aspects of its problems, the same firm that gave up the attempt of systematic study in disgust, could gradually build up an experience covering the whole range of its activities and acquire a fairly reliable basis for the interpretation of the results of loading tests. However, under the present system of interpretation, loading tests are practically good for nothing.

Distribution of Soil Reactions Over Rigid, Loaded Slabs

The preceding discussion deals exclusively with the potential settlement of structures without questioning the stresses that may develop within the foundation. As a matter of fact, when referred to individual spread footings, these questions are of minor importance. However, the situation changes when it becomes necessary to connect the footings with each other, replacing the group of footings by a continuous mat. In order to keep the costs of such a mat within reasonable limits, the stresses must be computed as closely

as possible. It is first necessary, however, to know the forces acting on the mat, and this is where difficulties arise. Suppose the mat is rectangular, and the dead load of the structure is distributed in such a manner that the resultant force passes through the center of gravity of the mat. Under such conditions, it is generally assumed, that the soil pressures are uniformly distributed over the entire area. However, according to the measurements carried out by M. L. Enger, M. Am. Soc. C. E., the distribution of the soil reactions over the base of a rigid slab is by no means uniform.* The pressures are equal to zero at the edge of the slab and greatest at the center, the pressure curve having a parabolic shape. In order to show the effect of the difference between uniform and parabolic stress distribution, the diagram, Fig. 3, has been plotted, showing the bending moments in a mat, loaded by four walls, and supported by a homogeneous soil. For a parabolic stress distribution the maximum bending moments will be about 100% greater than they are for a uniform reaction. Thus far, all attempts to deal theoretically with the problem of stress distribution have failed, and there is little hope for success within the near future. The data obtained by Mr. Enger are strictly confined to perfectly cohesionless materials; hence, the information they furnish is not yet conclusive. Considering the importance of the possible error involved in assuming uniform stress distribution, the need of a more exhaustive experimental investigation of this phase of the foundation problem becomes obvious.

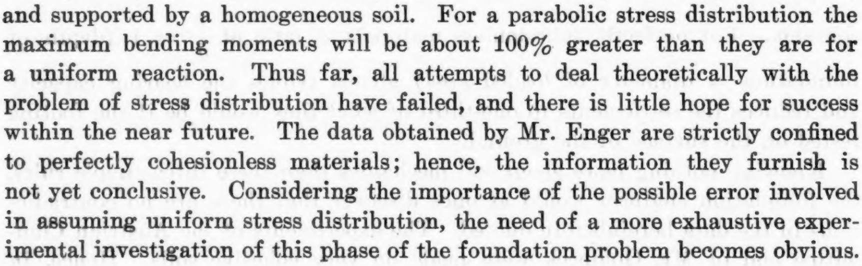

FIG. 3.—BENDING MOMENTS IN RECTANGULAR, PERFECTLY RIGID SLAB FOR (I) UNIFORM AND (II) PARABOLIC DISTRIBUTION OF THE SOIL REACTION.

EFFECT OF DEGREE OF PERMEABILITY AND OF TIME ON SETTLEMENTS

Another interesting aspect of the foundation problem concerns the changes that will take place in the soil under pressure. Since every soil, without exception, is compressible, the pressure tends to produce a decrease in volume.

* "The Distribution of Pressure by Granular Material," *Engineering*, Vol. 101, 1916, p. 170.

If the voids of the soil are filled with air, the volume change can take place at once, because the excess air can readily escape toward the surface. On the other hand, if the voids are completely filled with water, which is usually the case with very fine-grained and very compressible soils, (silts, mud, clay), a decrease in volume obviously involves a considerable decrease in the water content, and the compression cannot possibly proceed with a greater speed than the corresponding speed of squeezing out the excess water. The less permeable the soil, the more slowly the water escapes, and, as a consequence, the more slowly the volume of the compressed soil decreases. Hence, the settlement of the foundation will not occur at once. There will be a lag, depending on the degree of permeability of the soil.

The physical side of this gradual consolidation process has been thoroughly investigated, both theoretically and experimentally. According to theory, the consolidation of silts and coarse-grained muds should be accomplished within a couple of years, while for typical clays, it may be a couple of hundred years before the excess water has completely drained out. For each one of these examples, there are records accessible showing that the conclusions are correct. The rapid and thorough consolidation of silts under pressure is shown by the results of many test borings carried out by the Swedish Railroad Commission for the purpose of investigating the soil conditions under railroad dams.* From the physical tests performed with the drill samples, it was learned that the fills have produced a thorough, local consolidation of the soft strata supporting the surcharge. The consolidation was associated with considerable local subsidence. Fig. 4 shows a cross-section† of such a partly consolidated system. In contrast to this, the extreme slowness of consolidation of typical clays is illustrated by the following observation. In 1915, a building of the Massachusetts Institute of Technology was erected on a 30-ft. layer of sand, silt, and fill, resting on a soft clay deposit. The pressure exerted by the building on the clay amounts to approximately 1 500 lb. per sq. ft. In 1926, a test boring was made next to the heaviest section of the building. Physical examination of the drill samples disclosed the fact that the consolidation of the clay deposit had hardly started. A similar observation was published in 1923 by Thaddeus Merriman, M. Am. Soc. C. E., concerning the behavior of a mass of puddled clay forming part of an embankment on the Ashokan Reservoir, Catskill Water Supply.‡ The surface of the puddled clay deposit, confined between the core-wall and the up-stream slope of the core ditch, carries the weight of an embankment 85 ft. high. Nevertheless, nine years after the construction of the embankment, a test boring disclosed the fact that the clay core was practically in its original condition.

The study of these and similar facts has led to the recognition of two essentially different types of settlements:

* Statens Järnvägars Geotekniska Kommission, 1914–22, Stockholm, May, 1922.

† Copied from Staten Järnvägars Geotekniska Kommission, 1914–22.

‡ Discussion of the paper entitled "Design of Earth Dams," by Joel D. Justin, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 109.

(a) Those due mostly to lateral flow, with very little, or no, consolidation; and,

(b) Those due to consolidation and lateral flow combined.

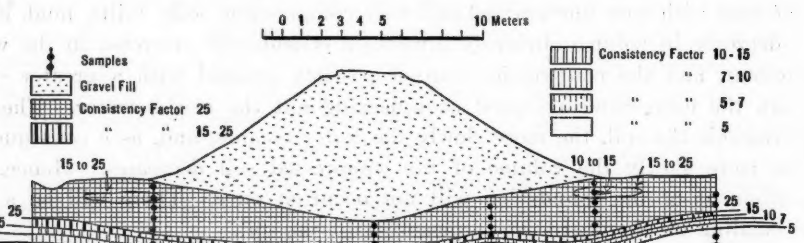

FIG. 4.—CONSISTENCY FACTORS OF SOIL DIRECTLY UNDER AND ON THE SIDE OF THE BANK AT SMEDSERÖD, SWEDEN.

In order to explain the characteristics of these two types of settlements, assume that two vertical reference lines, *a b* (Fig. 5), had been established in the ground, prior to the construction of the building. Under the influence of the weight of the building the soil tends to spread laterally, and the vertical lines, *a b*, become Curves *a c b*. The area included between the lines, *a b*, and the curves, *a b c*, is obviously equal to the area, $a\ a\ a_1\ a_1$, through which the building settles on account of lateral soil displacement. If, on account of a low permeability of the ground, the consolidation of the soil due to increased pressures, proceeds very slowly, the area, $a\ a\ a_1\ a_1$, represents practically the entire vertical displacement that the building will undergo, at least for one generation. The speed of the settlement will be governed exclusively by the laws of viscous flow in plastic materials, and if the flow is successfully stopped by driving deep sheet-piling, or by similar measures, the settlement of the building will be practically stopped. Those are the characteristics of Type (a) settlements.

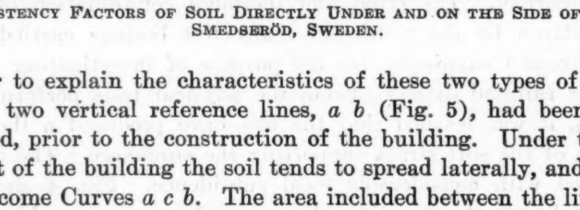

FIG. 5.—THE TWO PRINCIPAL SOURCES OF SETTLEMENT.

On the other hand, if the material is permeable enough to release its excess water within a couple of years, the settlement of the building will consist of two different parts, namely, the settlement through a space, $a\ a\ a_1\ a_1$, due to lateral bulging; and a settlement through an additional space, $a_1\ a_1\ a_2\ a_2$, due to compression of the soil beneath the loaded area. In this case the speed of the settlement will be governed by two essentially different laws: (1) Viscous flow in plastic materials; and (2) hydrodynamic stress compensation; which the writer formulated several years ago.* By artificially preventing the lateral flow of the material, one could somewhat reduce the settlements of the building, but one could not possibly prevent them. Those are the characteristics of Type (*b*) settlements.

The investigation of physical factors that determine the relation between time, consolidation, and deformation (lateral bulging and lateral flow), has reached a point where it is already possible to predict whether the settlements of a building will be of Type (*a*) or Type (*b*). In certain cases it is possible to estimate in advance the speed of the settlement. The knowledge of these factors is particularly useful in all cases where the engineer faces the problem of reducing or stopping the settlement of an existing building.

Bearing Capacity of Individual Piles

At the beginning of this paper it was assumed, that the engineer trusted with the design of a building, decided to erect it on spread footings. Let it now be assumed that his decision was premature. Suppose that test borings made after the first inspection of the building lot revealed the fact that the solid top layer of soil rested on a deposit of soft silt of such a thickness that there was no chance to carry the foundations down to solid ground. Then the engineer decided to put the foundations on piles. In order to find out how many piles he needed, he was obliged to drive a couple of test piles and determine the amount of load one individual pile could stand (unless he had previously tested some piles driven into similar deposits). To follow the traditional procedure, one must observe the average penetration of the pile under the last ten blows; compute the bearing capacity of the pile by the *Engineering News* formula; and divide the total weight of the building by the bearing capacity of the individual pile. This practice rests on two assumptions: (1) That the bearing capacity of a pile can be computed from the effect of the blow; and (2) that the bearing capacity of the complete foundation is equal to the sum of the bearing capacities of the individual piles. To examine Assumption (1), let,

R = weight of the hammer.
G = weight of the pile.
L = length of the pile.
F = area of the cross-section of the pile.
E = modulus of elasticity of the pile material.
h = distance the hammer drops.
s = penetration produced by one blow.

* "Die Theorie der Hydrodynamischen Spannungserscheinungen und ihr erdbautechnisches Anwendungsgebiet," *Proceedings*, International Congress for Applied Mechanics, Delft, Holland, 1924.

m = coefficient of elasticity of the impact; $m = 0$ for perfectly non-elastic impact; and $m = 1$ for perfectly elastic impact.

Q_d = resistance against penetration of the pile, under impact.

Q = ultimate bearing capacity of the pile under static load.

C = empirical constant depending on the nature of the pile and the resistance against penetration $= \frac{Q_d}{2} \frac{L}{F E}$

The theory of semi-elastic impact leads to the following equation:

$$Q_d = \frac{F}{L} E \left[- s^2 \pm \sqrt{s^2 + \frac{2 R h}{E} \frac{R + m^2 G}{R + G} \frac{L}{F}} \right] \ldots\ldots\ldots\ldots (3)$$

The value, m, is usually assumed equal to 0.5 (semi-elastic impact). For $m = 0$, the equation becomes Redtenbacher's formula, which is quite extensively used in Europe.* On the other hand, if perfect elastic impact is assumed, $m = 1$, Equation (3) becomes,

$$R h = Q_d + \frac{1}{2} \frac{Q_d^2}{F} \frac{L}{E} = Q_d \left(s + \frac{1}{2} \frac{Q_d}{F} \frac{L}{E} \right) \ldots\ldots\ldots\ldots (4)$$

or,

$$Q_d = \frac{R h}{s + \frac{1}{2} \frac{Q_d}{F} \frac{L}{E}} \ldots\ldots\ldots\ldots (5)$$

If the fact that the term, $\frac{1}{2} \frac{Q_d}{F} \frac{L}{E}$, depending on both the nature of the pile and the resistance against penetration, is disregarded, and if this variable term is replaced by an empirical constant, C, independent of all these factors,

$$Q_d = \frac{R h}{s + c} \ldots\ldots\ldots\ldots (6)$$

which is none other than the well-known *Engineering News* formula.

From a mechanical point of view, the assumptions on which these formulas are based, are sound, and there is not much doubt about their giving a fairly accurate conception of the resistance one has to overcome while driving the pile by a succession of impacts. Therefore, if experience shows that in certain cases the values furnished by the pile-driving formulas have practically nothing in common with those determined by loading tests, being either far too small or far too large, the cause cannot be a defect in the formulas, but must be due to the fact that in these particular instances the forces, Q_d, resisting the penetration of the pile under impact, are fundamentally different from the forces, Q, which resist the penetration of the pile under static load.

This is precisely the situation a theoretical study of the pile-driving phenomenon has disclosed. It is known that the bearing capacity of piles depends on two different factors, namely, the frictional resistance acting along the sides of the piles, and the point resistance, or the resistance of the soil against being compressed and displaced by the pile. If these two resistances

* "Formeln und Versuche über die Tragfähigkeit eingerammter Pfähle," Th. Krapf, Leipzig, 1906.

were dependent only on the character of the ground and on nothing else there could be no question about the resistance, Q_d, against driving the pile, and the bearing capacity, Q, of the pile, being identical. However, it is easy to prove that either one of them may be very different according to whether the pile is slowly forced down or driven by impact.

If a friction test is made on a layer of sand, by loading it and then measuring its resistance against shear, it is found that the shearing resistance of the loaded layer, immediately after the application of the load, is practically the same as it is three days later. If, however, precisely the same test is performed with a layer of clay immersed in water, it is found that immediately after application of the load the frictional resistance is very small, so small indeed that one has the impression that the material is lubricated. The full frictional resistance does not develop for a couple of days. If a laterally confined mass of sand is compressed, the speed with which the compression is performed has very little influence on the amount of work required to compress the material. On the other hand, the amount of work required for rapidly reducing the volume of 1 cu. ft. of laterally confined clay by 2 cu. in. may be 100 times as great as the amount of work required for producing the same volume change slowly. The physical causes of these phenomena are clearly understood. The writer has called them the hydrodynamic stress phenomena. They inevitably develop as a result of rapid application of loads or pressures on water-soaked materials with a low degree of permeability.* The theory has been repeatedly checked by experiment.

Applied to the mechanics of pile-driving, knowledge of the hydrodynamic stress phenomena has led to classifying soils in two main types. In certain materials (particularly in sand, gravel, and permeable artificial fills), the resistances acting while the pile is being driven, are practically identical with those acting on the pile under static load. Under such conditions the pile-driving formulas can be expected to furnish results of sufficient accuracy.

In other materials (very fine-grained silts, soft clays, etc.), the friction acting on the pile during the driving (hydrodynamic pile friction) is very much less than that which develops after a couple of days' rest (static pile friction), while the resistance of the point of the pile under impact (dynamic point resistance) is very much greater than its resistance under static load (static point resistance). Due to these facts the total resistance against penetration of the pile into such materials is:

(1) Dynamic resistance (resistance, Q_d, against penetration under impact), which is the sum of a very small frictional resistance (dynamic pile friction) and a very considerable point resistance (dynamic point resistance).

(2) Static resistance (ultimate bearing capacity, Q, under static load), which is the sum of a full frictional resistance (static pile friction) and a very small point resistance (static point resistance).

Since the pile-driving formulas furnish the value, Q_d, there is no assurance whatsoever that for this class of materials the value, Q, may be of the same order of magnitude. The value, Q_d, may, by chance, be equal to Q, the

* "Erdbaumechanik," by Charles Terzaghi, Wien, 1925.

deficiency in static friction being compensated by an excess in point resistance; but this condition is by no means necessary. It could as well be very much greater or very much less, depending on the material. The following analogy demonstrates the error committed when applying the pile-driving formulas to resistance against the penetration of piles in this second class of materials. Suppose that a body slides on a rail under water. In order to produce a slow forward movement of this body, the only resistance there is to overcome is the static friction between the body and the rail, the resistance of the water being negligible. On the other hand, in order to keep the body sliding rapidly along the rail, the only resistance that counts is the resistance of the water, which, in this case, may be very considerable, while the frictional resistance along the rail will practically be eliminated because of a film of water trapped between the rail and the sliding body. Application of a pile-driving formula to materials of the second class is no more logical than an attempt to identify the static resistance of the sliding body against being set in motion with its dynamic resistance acting while moving rapidly through a viscous medium.

The best way to distinguish whether a material belongs to the first or to the second class is that of comparing the penetration per blow immediately before and after a period of rest. If these two penetrations are identical, one can be quite sure that the material belongs in the first class, and that the pile-driving formulas can be expected to furnish fairly reliable results. However, for this case experience seems to show that the formulas based on the theory of impact (Equation (3)), furnish far better results than the *Engineering News* formula.

In 1925, the writer investigated the bearing capacity of an artificial fill, consisting of residual soil (stones, sand, and earth mixed), at Pascha Liman, on the Asiatic shore of the Bosphorus. When an attempt to make test borings with a normal outfit failed, several test piles were driven, one of which was loaded to the limit of its bearing capacity. The pile-driver was of the drop-hammer type with a weight of 0.575 metric tons. Fig. 6 (*a*) shows the results of the loading test. Fig. 6 (*b*) shows the resistance against penetration under impact computed by the *Engineering News* formula (dotted lines) (Equation (6)) and by the theory of semi-elastic impact (thin full drawn line) (Equation (3)). The two sets of values are not very different, because the penetrations were rather important (ranging between 1.0 and 1.4 in.), although the values obtained by the impact theory are nearer the ultimate bearing capacity determined by the loading tests.

However, with decreasing values of the penetration produced by the last blows, the error involved in the *Engineering News* formula rapidly increases. A rather striking example can be quoted from pile-driving in Germany. As a result of many years of experience, the foundation engineers of Berlin, Germany, have developed the following empirical rule for the sandy soil of that city. For a pile that penetrates less than 0.4 in. under the impact of a 1-ton hammer, dropping from an elevation of 3.3 ft., the ultimate bearing capacity ranges between 20 and 25 tons. If the *Engineering News* formula is applied to this case, the ultimate bearing capacity is found to be 28.2 tons,

corresponding to an allowable load of $\frac{1}{6} \times 28.2$ tons = 4.7 tons! For the ultimate bearing capacity of the same pile, Redtenbacher's formula* furnishes a value ranging between 52.8 to 63.6 tons, which is very close to the actual ultimate bearing capacity of these piles.

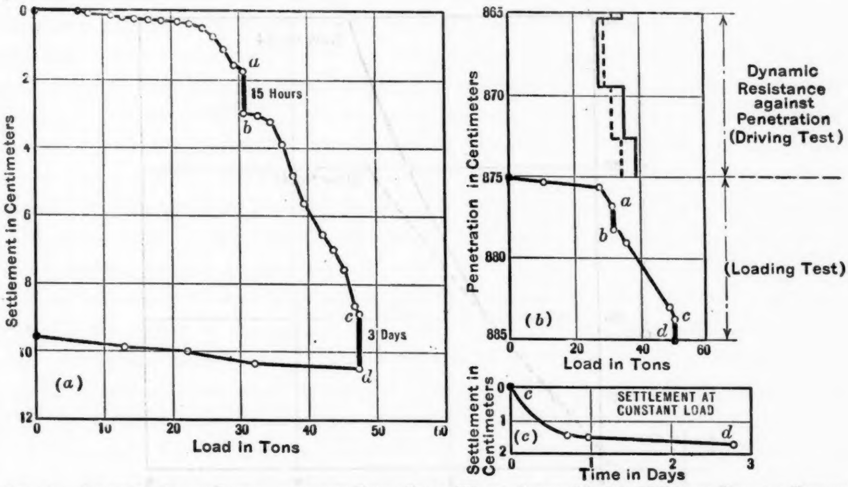

FIG. 6.—LOADING TEST PERFORMED ON TEST PILE IV ON APRIL 20–24, 1925, AT PASCHA LIMAN, ASIA MINOR.

If applied to impact, associated with small penetration, the defects of the *Engineering News* formula become particularly obvious. In connection with the pile tests performed in Pascha Liman, penetrations under the influence of a series of blows were observed, the hammer dropping from an elevation of 6.57 ft. (2 blows), 1.64 ft. (2 blows), 3.30 ft. (2 blows), 6.57 ft. (4 blows), and 9.83 ft. (4 blows). Fig. 7 shows the results of the observations. The points that correspond to a drop of the hammer of 6.57 ft. (200 cm.), 1.64 ft. (50 cm.), and 3.30 ft. (100 cm.), are located on a straight line. According to the theory of pile-driving by impact, this fact indicates that the resistance of the ground remained unchanged during the driving period. Hence, for this period, a correct pile-driving formula ought to furnish identical values, irrespective of the height from which the hammer was dropped. Table 1 shows the values obtained by the *Engineering News* formula and the theory of semi-elastic impact, respectively.

Table 1 demonstrates that for penetrations of less than 1 in., the *Engineering News* formula furnishes values that are by far too small, the error rapidly increasing with decreasing depth of penetration.

The obvious reason for this deficiency of the *Engineering News* formula is that the variable item, $\frac{1}{2}\frac{Q_a}{F}\frac{L}{E}$, (Equation (5)), forthcoming as a result of the theory of impact, has been replaced by a constant, C.

* "Formeln und Versuche über die Tragfähigkeit eingerammter Pfähle," by Th. Krapf, Leipzig, 1906.

All these facts and data refer to typical materials of the first class, that is, to materials for which the pile-driving formula can be expected to furnish fairly reliable values. This conclusion was reached by observing, among other things, that the penetration produced by impact of a given intensity was practically the same before and after a period of rest.

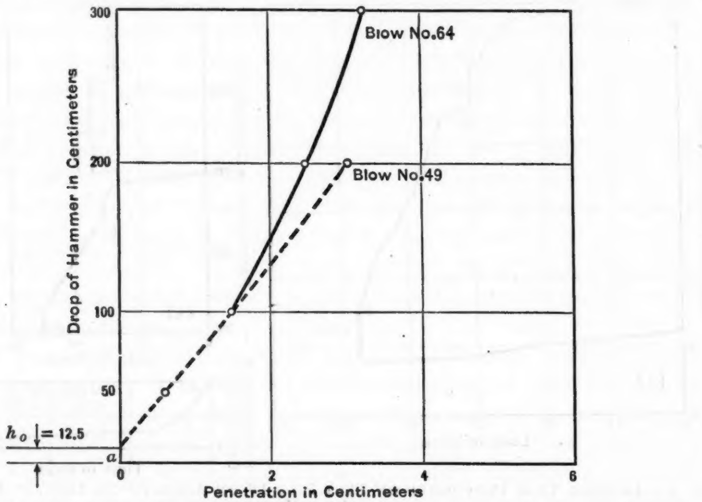

FIG. 7.—RELATION BETWEEN THE DROP, h, OF THE HAMMER AND THE CORRESPONDING PENETRATION, s, PRODUCED BY THE BLOW.

On the other hand, if the penetrations produced by a given impact before and after a period of rest are very different, the material belongs in the second class, and the values furnished by any pile-driving formula may be either equal to, very much larger than, or very much smaller than, the static bearing capacity of the pile.

TABLE 1.—COMPARISON OF THE BEARING VALUE OF PILES.

Drop of hammer, in feet.	Penetration, in inches.	ULTIMATE BEARING CAPACITY, IN TONS, BY: *Engineering News* formula.	Semi-elastic impact theory.	Remarks.
6.57	1.160	21.0	25.4	Should be equal
1.64	0.236	9.2	26.5	
3.30	0.590	14.2	24.6	
6.57	1.160	23.0	30.3	....
9.83	1.280	29.7	35.2	....

Notice the physical processes associated with driving a pile into a material of the second class. Every blow of the hammer squeezes a certain quantity of water out of the soil beneath the point of the pile. The water escapes

toward the surface through the space between the pile and the ground, and forms a film acting as a lubricant, just as any liquid does if trapped between two surfaces. Due to the presence of this film, the friction acting along the surface of the pile is very much less, whereas the force required to squeeze the water rapidly out of the soil beneath the point of the pile is greater than that required to compress the same soil slowly. During a period of rest, the film of water is gradually absorbed by the soil and the full static pile friction develops. When pile-driving is resumed, one has to overcome both the static friction and the dynamic point resistance. However, during the same period of rest the annular space, serving as an outlet for the water squeezed out of the soil by impact, has closed. Hence, in certain kinds of soils, the dynamic point resistance acting after a period of rest may be much more than such resistance acting before the rest.

The following figures may serve to illustrate an extreme case of this type. The soil consisted of a succession of layers of soft loam, some of them mixed with vegetable fibers (peat). The penetration observations, the loading tests, and the pulling tests were made, in 1904, by Mr. Th. Krapf in connection with the construction of a bridge across the Rhine Valley Canal in Austria.* The dynamic pile-driving resistance has been computed from the penetration data by the theory of semi-elastic impact ($m = 0.5$) (see Equation (3)). The pile was 8.4 m. long; lower diameter, 23.6 cm.; upper diameter, 33.3 cm.; and weight of the hammer, 765 kg. (drop-hammer). The data concerning this pile were, as follows: Ultimate bearing capacity (loading test), 17.2 tons; skin friction (pulling test), 14.4 tons; static point resistance, 17.2 — 14.4 tons = 2.8 tons; dynamic pile-driving resistance, computed from penetration after continuous driving, 20.0 tons; and dynamic pile-driving resistance, computed from penetration after a period of rest of 30 days, 80.0 tons.

In this case, by mere chance, the pile-driving formula applied to the effect of the hammer for continuous driving furnished a value close to the actual bearing capacity; while the value derived from the penetration after a period of rest was by far too great. In other cases, the second value is found to be closer to the actual bearing capacity. This may be learned from the customary practice of introducing into the *Engineering News* formula the values obtained after a period of rest.

Based on what is known about the physics of the penetration of a pile in the second class of material, the aforementioned data can be interpreted. While continuously driving the pile into the ground, the skin friction is practically eliminated. Hence, the dynamic pile-driving resistance is practically equal to the dynamic point resistance; or, before the period of rest, the dynamic point of resistance is 20.0 tons, as compared with 2.8 tons static point resistance.

After a 30-day period of rest, the hammer had to overcome the full static friction (17.2 tons) plus the dynamic point resistance (62.8 tons); or a total of 80 tons.

* "Formeln und Versuche über die Tragfähigkeit eingerammter Pfähle," by Th. Krapf, Leipzig, 1906.

Fig. 8 represents this interpretation graphically. The shaded areas correspond to the pile friction acting during the pile-driving process (*a*); under impact after a period of rest (*b*); and under static load (*c*).

This and similar examples show that the application of any pile-driving formula to the bearing capacity of piles driven into the second class of materials is a gamble, trusting that the deficiency in skin friction associated with the driving of the pile may, by chance, be compensated by the corresponding excess in point resistance.

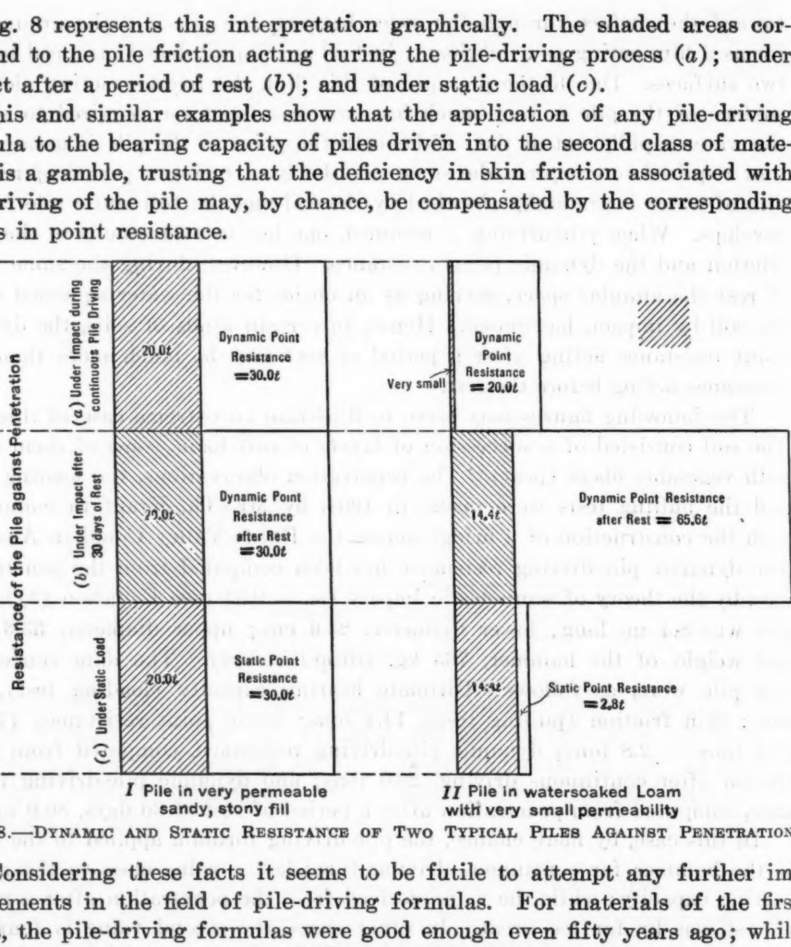

FIG. 8.—DYNAMIC AND STATIC RESISTANCE OF TWO TYPICAL PILES AGAINST PENETRATION.

Considering these facts it seems to be futile to attempt any further improvements in the field of pile-driving formulas. For materials of the first class, the pile-driving formulas were good enough even fifty years ago; while for materials of the second class, no reliable pile-driving formulas are possible at all, because the resistance against penetration of the piles under impact into such materials has physically nothing more in common with the static bearing capacity than has the static friction between solids (following the law of Coulomb) with the viscous resistance of liquids against rapid deformation (following the law of Newton). In the future, the penetration curve of piles, and particularly the difference between the rate of penetration before and after a period of rest, may give more reliable information about the character of the ground than a test boring, but it never will give any reliable information about the bearing capacity of the piles.

Therefore, future investigations in the field of bearing capacity of piles should be directed toward studying the effect of the shape, thickness, and length on the bearing capacity of piles driven into different kinds of soils. In this connection attention should be called to the exhaustive investigations by the engineers of the Whangpoo Conservancy Board, in Shanghai, China.

The tests included more than forty complete driving, loading, and pulling tests in soils the nature of which was previously investigated by test borings. The soil consisted principally of clay and silt, apparently belonging to the second class of materials. The tests furnished valuable information concerning the effect of the shape and length of the piles on the ultimate bearing capacity. Characteristically enough, the published reports do not contain any information on the penetration observations, for the reason that "the application of general pile-driving formulas to the local conditions has resulted in great errors".*

Bearing Capacity of a Pile Foundation

From the preceding considerations it may be seen that Assumption (1) on which present methods of planning pile foundations are based, may or may not be admissible, according to the character of the material. This situation is not serious because, if it becomes necessary to exclude Assumption (1) (that the bearing capacity can be computed from the effect of the blow), the required data can be obtained by making a loading test.

Far more important is the second assumption, (2), concerning the relation between the behavior of the individual pile and the pile foundation as a whole. When discussing this assumption, it is best to distinguish between three different cases: (1) The piles transfer the weight of a building to bedrock or to another stratum, the bearing capacity of which is equal to or greater than the load acting per unit of the cross-sectional area of the pile; (2) the piles transfer the weight of the building through a very compressible top layer to a less compressible one; and (3), the piles are driven for a certain depth into a deep deposit, the consistency of which does not appreciably increase with the depth.

Case (1) hardly needs any discussion. In Case (2) there are two different possibilities; either the feebly resistant top layer can or cannot be solidified by pile-driving. For instance, the average water content of the soft clay deposits in Boston, Mass., ranges between 30 and 40%, corresponding to a volume of voids ranging between 45 and 52%, filled with water. If piles are driven into a deposit of this kind, the surface of the clay rises between them through a height of several inches, which indicates that the volume of voids of the material remains practically unchanged. Pile-driving produces not only no consolidation of the deposit, but it even seems to cause the material to become softer. This last conclusion has been drawn from the observation that the compressive strength of an undisturbed sample of clay is always considerably greater than that of the same sample, with the same water content, after the sample has been moulded by external pressure. In striking contrast to this behavior of the deposits of blue clay stands the behavior of other very fine-grained deposits. Quite recently, when piles were driven into a deposit of exceedingly fine-grained saturated quicksand, the surface of the deposit subsided between the piles through a distance of almost 1 ft. Judging from

* Whangpoo Conservancy Board, S. H. T. Series 1, No. 7. Various reports to the Engineer-in-Chief on special investigation, Shanghai, 1921; report to the Engineer-in-Chief on pile tests.

previous experience, the original volume of voids of the material was certainly not greater than 45%, corresponding to the volume of voids of a clay with a water content of 30 per cent. Nevertheless, judging from the subsidence of the surface of the deposit, the pile-driving must have reduced the volume of voids by several per cent., which, in turn, involves a considerable increase in the bearing power and a decrease of the compressibility of the deposit.

In these two cases, the piles must serve two very different purposes if they are to be utilized to full advantage. In the first case, they should transfer the load to the more resistant sub-stratum, thus diverting the pressure from the upper to the lower stratum (Fig. 9 (*a*)). In order to serve this purpose, the bearing capacity of the section, *b c*, of the pile below the bottom of the upper layer must be great enough to support the load without any appreciable settlement. Since the result of a loading test performed on an individual pile includes both the bearing capacity of this section of the pile and the skin friction acting along the section, *a b*, this result may be misleading.

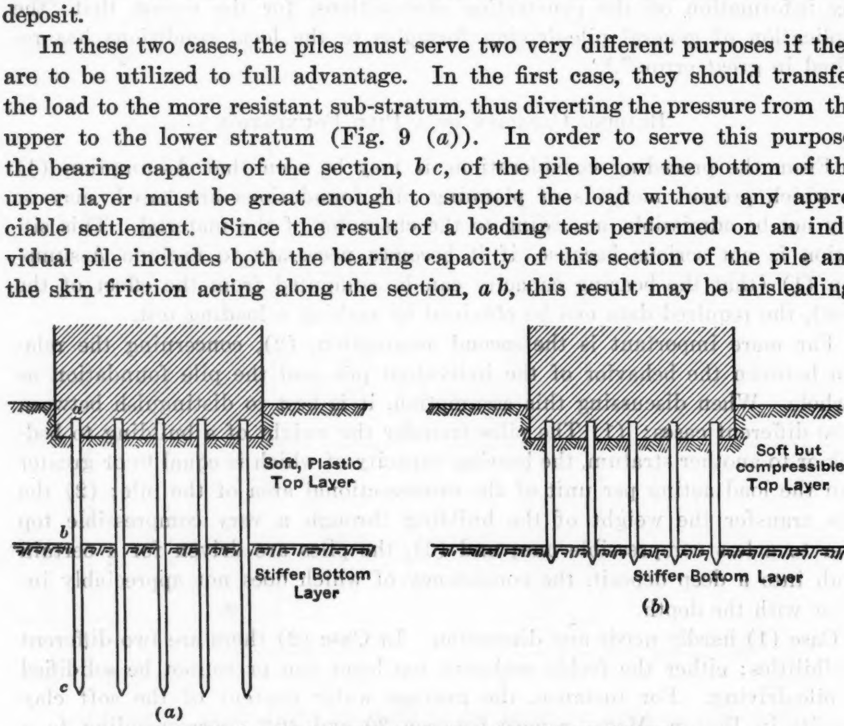

FIG. 9.—PILE FOUNDATION ON A TWO-LAYER SYSTEM.

Fig. 9 (*b*) represents the same combination of strata as Fig. 9 (*a*), with the only difference that the top layer is apt to be solidified (artificial fill, quicksand, etc.). In this case it may be more economical to use the piles merely for the purpose of increasing the bearing capacity of the top layer by pile-driving, letting the piles extend to the bottom of the top layer only. Here, again, the result of a loading test performed on an individual pile, prior to driving the others, would fail, by far, to furnish any reliable information concerning the number of piles required. What is needed is to learn about the effect of driving on the density of the surrounding soil. Deposits that can be consolidated by pile-driving belong almost exclusively to those classes of material for which the pile-driving formulas are valid. The proper procedure would be to start with piles far apart and to drive intermediate piles until the bearing capacity, computed from the penetration, indicates the resistance of a well-consolidated ground.

In the third case (piles driven into the upper part of a very deep deposit with a fairly uniform consistency), the value of the piles may or may not be problematical, depending on the ratio between the width of the foundation and the length of the piles. Suppose that the pressure diagrams, Fig. 10 (*a*) and (*b*), have been plotted. The left-hand sections of these diagrams show the distribution of soil pressures beneath two raft foundations with different widths, computed approximately by Boussinesq's theory. The right-hand sections of the same diagrams show the change in the stress distribution due to the presence of 20-ft. piles in the ground, computed by the same theory.

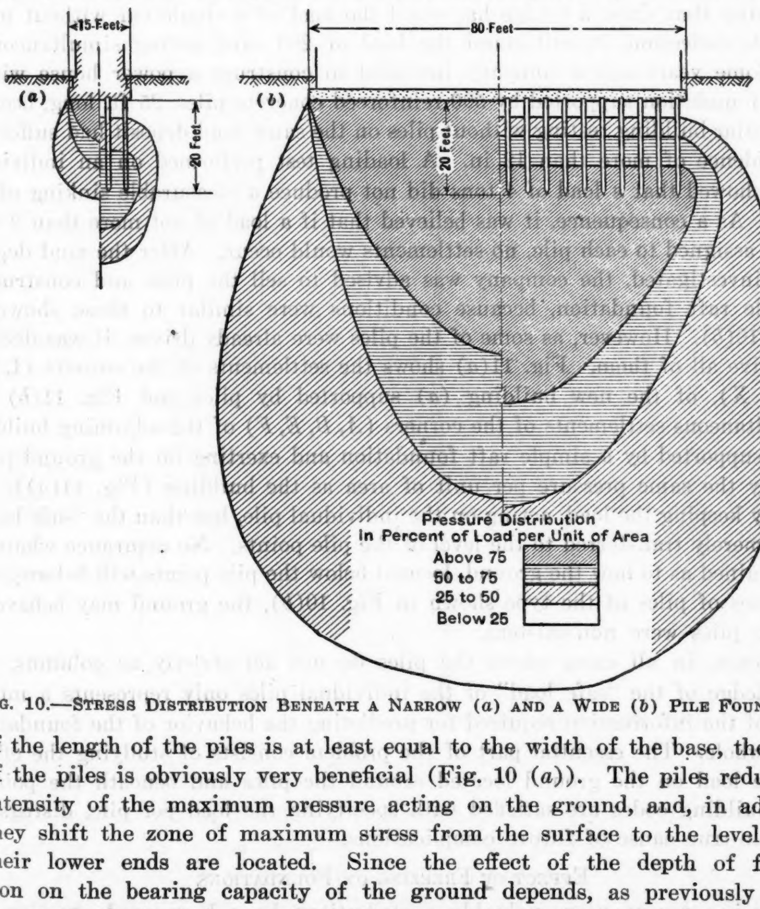

FIG. 10.—STRESS DISTRIBUTION BENEATH A NARROW (*a*) AND A WIDE (*b*) PILE FOUNDATION.

If the length of the piles is at least equal to the width of the base, the effect of the piles is obviously very beneficial (Fig. 10 (*a*)). The piles reduce the intensity of the maximum pressure acting on the ground, and, in addition, they shift the zone of maximum stress from the surface to the level where their lower ends are located. Since the effect of the depth of foundation on the bearing capacity of the ground depends, as previously noted, not on the depth, *t*, but on the ratio between the depth of foundation and its width, the effect of transferring the pressure to a deeper level, in the case of Fig. 10 (*a*), may be quite important. On the other hand, if the width of the foundation is considerably greater than the length of the piles (Fig. 10 (*b*)), the pressure-reducing effect of the piles is very small and the ratio between the depth of foundation and the width of the structure is also very much smaller than in the case of Fig. 10 (*a*). Hence, the beneficial effect of the piles may

be negligible and the money invested in them may represent an unwarranted expenditure. The reason piles are so generously used beneath raft foundations, in spite of the facts represented in Fig. 10, is that many engineers confound the bearing capacity of the individual piles with the bearing capacity of a score of them. Yet one can hardly conceive of a more obvious fallacy. The individual pile spreads the load over a wide area, thus reducing the specific soil pressure to a negligible item, while beneath a wide pile foundation, the soil pressure is just as great as it is under a simple raft foundation of equal width. In structural engineering such a procedure would correspond to considering that since a bridge has stood the load of a single car without measurable deflection, it will stand the load of 200 cars, acting simultaneously.

Some years ago a company intended to construct a power house with a raft foundation supported by 500 reinforced concrete piles, 25 ft. long, because a similar building resting without piles on the same mud deposit had suffered a subsidence of more than 12 in. A loading test performed on an individual pile showed that a load of 4 tons did not produce a measurable sinking of the pile. As a consequence, it was believed that if a load of not more than 2 tons were assigned to each pile, no settlements would occur. After the mud deposit was investigated, the company was advised to sell the piles and construct a simple raft foundation, because conditions were similar to those shown in Fig. 10(*b*). However, as some of the piles were already driven, it was decided to drive all of them. Fig. 11(*a*) shows the settlements of the corners (I, IV, VII, X), of the new building (*a*) supported by piles and Fig. 11(*b*) the simultaneous settlements of the corners (A, B, E, F) of the adjoining building (*b*), supported by a simple raft foundation and exerting on the ground practically the same pressure per unit of area as the building (Fig. 11(*a*)).

By keeping the load, acting on the individual pile, less than the "safe load", it is merely transferred to the level of the pile points. No assurance whatever is obtained as to how the ground, located below the pile points will behave, and in cases of piles of the type shown in Fig. 10(*b*), the ground may behave as if the piles were non-existent.

Hence, in all cases where the piles do not act strictly as columns, the knowledge of the "safe load" of the individual piles only represents a minor part of the information required for predicting the behavior of the foundation as a whole. The essential part of the problem consists of studying the effect of the load on the ground located around the piles and beneath the points. Yet building codes are satisfied with specifying the load per pile, disregarding the true cause of future complications.

Effect of Freezing on Foundations

During recent years valuable contributions have been made to current knowledge of the effect of frost in the soil and on the structures supported by it. Most important among them are the investigations of Professor Stephen Taber,* of the University of South Carolina, concerning the capacity of freezing veins of water to absorb additional water out of the surrounding

* "The Growth of Crystals Under External Pressure," *American Journal of Science*, 4th Series, Vol. 41, pp. 532–556; "Pressure Phenomena Accompanying the Growth of Crystals," *Proceedings*, National Academy of Sciences, Vol. 3, No. 4, pp. 297–302.

clay soil, or through the clay soil from the ground-water, and to develop from thin seams into layers with a considerable thickness. The space required for the growth is produced by the pressure of crystallization, forcing the soil out of the way. Additional laboratory and field investigations are planned by the U. S. Bureau of Public Roads in co-operation with Professor Taber and the writer.

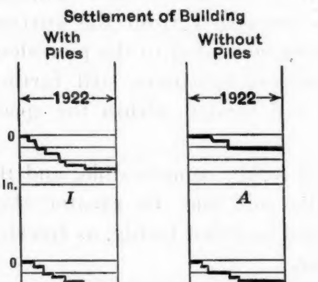

FIG. 11.—SETTLEMENT OF TWO ADJOINING BUILDINGS.

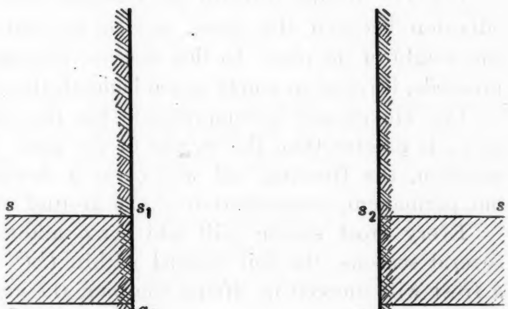

FIG. 12.—EFFECT OF SOIL FREEZING ON A FOUNDATION PIER.

In connection with foundation engineering, the importance of frost action essentially resides in the capacity of the freezing ground to lift foundations bodily several inches above their original position. This is apt to cause differential heaving associated with secondary stresses in the superstructure. Quite a number of such cases have been brought to the attention of the profession and many more have been undoubtedly observed.*

* "Freezing Ground Acts Like Hydraulic Jack: Heaving and Settling Back of Piers of Railroad Bridge Over City Street, Lead to Interesting Deduction," by H. J. Gilkey, *Engineering News-Record*, Vol. 79 (1917), pp. 360–361; "Some Observations on Effect of Frost in Raising Weight," by L. B. Wyckoff. *Engineering News-Record*, Vol. 80 (March 28, 1918), pp. 627–628; "Water Expansion in Ground Cause of Heaving in Winter," by C. D. Norton, *Engineering News-Record*, Vol. 80 (May 30, 1918), p. 1058; "Action of Frost in Heaving Concrete Piers," by L. DeB. McCready, *Engineering News-Record*, Vol. 91 (August 30, 1923), p. 360.

However, thus far, apparently no attempt has been made to go beyond the observed phenomena and to analyze the conditions required for producing the lifting effect. The following conclusions represent the results of a preliminary survey of these conditions.

The freezing starts at the surface of the ground and gradually proceeds toward the interior. At the same time the soil freezes to the pier, the adhesion being practically equal to the shearing strength of the frozen soil. Suppose that at some intermediate state, freezing has proceeded from the surface, *S-S* (Fig. 12), to the level, *aa*, the frozen soil being cemented to the pier along the faces, $a_1\ s_1$ and $a_2\ s_2$. If the freezing process continues still farther, causing the freezing and the expansion of the soil located within the space, *a a b b*, there are then three possibilities:

(*a*) The ground beneath the freezing layer is feebly compressible, and the adhesion between the faces, $a_1\ s_1$, $a_2\ s_2$, and the soil may be greater than the weight of the pier. In this case the pier should be lifted bodily, as freezing proceeds, leaving an empty space beneath the pier.

(*b*) The ground is compressible, but the adhesion along the faces, *a, s,* and $a_2\ s_2$, is greater than the weight of the pier. In this case, with the pier as a reaction, the freezing soil will exert a downward pressure causing a small, but permanent, consolidation of the ground within the spaces, *C-C*.

Every frost season will add some additional consolidation, until, after several seasons, the soil located within *C-C* will be so compact, that finally a frost will succeed in lifting the pier, the consolidated material, *C-C,* acting as a reaction.

(*c*) The ground is feebly compressible, but the weight of the pier is greater than the adhesion. In this case the expansion of the soil, *a a b b*, will cause a slow upward creep of the soil, *s s a a,* along the outside of the pier, associated with an upward lift equal to the adhesion. The result should be a fatigue effect, similar to repeated application and removal of a live load, causing slight additional settlements of the pier.

From these remarks it may be learned that the lifting action of the expanding ground should depend on several factors, namely, the compressibility of the unfrozen foundation soil, the compressive strength of the frozen soil, the depth of freezing, and, finally, on the ratio between the weight of the pier and the total adhesion between the pier and the frozen ground. As experiences increase, other factors may have to be added to this list. Considering the scarcity of available information, any observations made in this field may increase the engineer's capacity for predicting the effect of freezing on proposed foundations.

Soil Classification Based on Elastic Constants of Soils

The preceding parts of this paper have brought out the following facts: (1) The settlement produced by a given unit load may increase either in direct proportion with the diameter of the loaded area, or at a very much smaller rate, depending on the character of the soil; (2) the settlement of a building may be due to volume change combined with lateral flow, or to

lateral flow alone, depending on the character of the soil; (3) the pile-driving formulas may furnish fairly reliable or utterly inconsistent results, depending on the character of the soil; and (4) the driving of piles into a soft top layer may cause a softening of the soil associated with a rise of the surface, or consolidation associated with subsidence, depending on the character of the soil.

The fundamental requirement for bringing the manifold foundation experience into a rational working system consists of establishing a system for the classification of soils based essentially on those characteristics that are of engineering importance.

Thus far, attempts to classify soils (including the revised soil classification scheme of the Special Committee on Soils for Foundations, etc., of the Society) have been based essentially on such properties of soils as: (*a*) mineral composition; (*b*) volume of voids; (*c*) grain composition (result of mechanical analysis); (*d*) water content; and (*e*) percentage of colloidal material present in the soil. A study of these factors, covering a period of several years, has disclosed that:

(*a*) The mineral composition of very fine-grained soils cannot be determined except by elaborate optical or chemical methods, the cost of which is forbidding. Even if it were possible to make such a determination, the benefit would be doubtful.

(*b*) The volume of voids is so complicated a function of the shape and uniformity of the grains that it is impossible to correlate it with any definite properties of the soil, even if both the uniformity and the effective size of the material are known.

In 1926, the writer investigated the permeability of two sets of samples; one coming from a deposit of modified glacial drift near Westfield, Mass., and the other from a glacial lake deposit near Springfield, Mass. The effective size of both materials ranged between 0.01 mm. and more than 1 mm. In order to simplify the laboratory work, an effort was made to determine for each set the relation that exists between the uniformity coefficient (according to the well known definition of Allen Hazen, M. Am. Soc. C. E.) and the volume of voids. Fig. 13 shows the results of this investigation. Although there was a slight difference only between the shape of the grains and the mica content of the various samples, the relation between the volume of voids and the uniformity coefficient was found to be very erratic. To quote an example: At a voids' ratio of 0.8 (volume of voids of 45%), Soil No. 7, from Westfield, is as compact as it could possibly be and, if loaded, it would have a considerable bearing capacity. At the same voids ratio of 0.8 (volume of voids of 45%), Soil No. 34, from the same locality, would be very compressible, although its uniformity coefficient is practically equal to that of Soil No. 7.

(*c*) A mechanical analysis, according to the revised size grades of the Society's Special Committee on Soils for Foundations, etc., involves more than two weeks of work per sample, and the costs are forbidding. Nevertheless, the writer performed more than one hundred complete mechanical analyses of

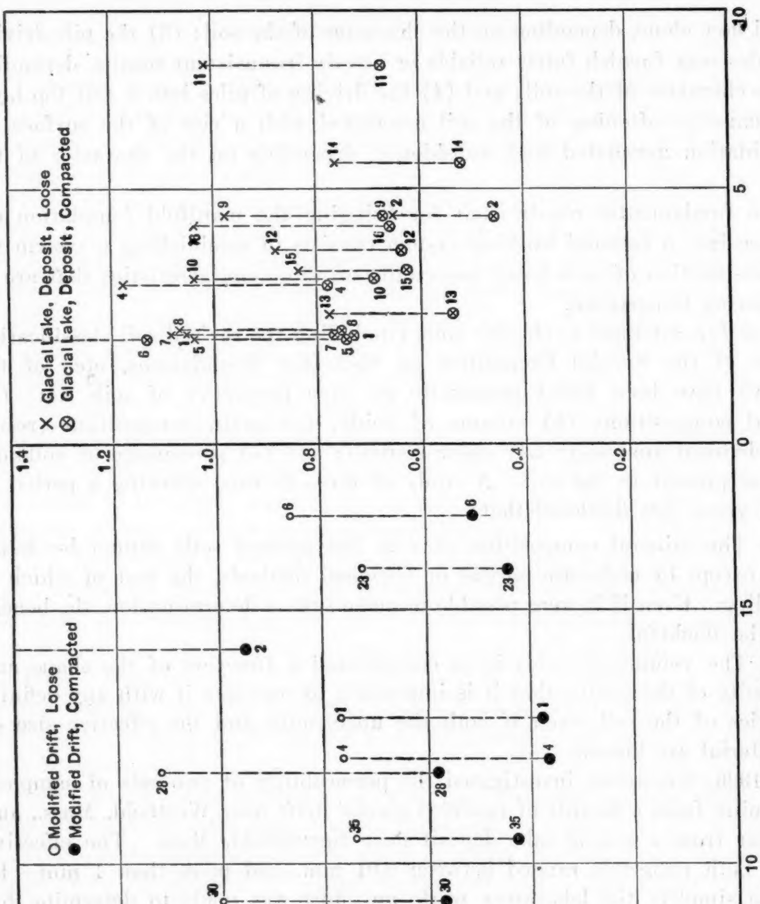

FIG. 13.—RELATION BETWEEN VOIDS RATIO AND UNIFORMITY.

soils of all types (super-centrifuge treatment excluded). As a result it was found that two soils with utterly different physical characteristics may have almost identical mass diagrams.

(*d*) The water content, too, seemed to be a very unreliable guide in judging the characteristics of soils. For example, it was found that some clays with a water content of 40% are considerably stiffer than others with one of 25 per cent.

(*e*) During the last few years, increasing attention has been paid to the colloids present in the fine-grained (cohesive) soils, and it has been hoped, to solve the problem of soil classification by finding some simple means of determining this percentage. These hopes apparently were based on the idea that the "soil colloid" is a specific and well-defined substance with very unusual properties, and that the knowledge of the percentage of colloids present in the soil should be sufficient to judge its character. The last International Soil Congress (June 1927, Washington, D. C.), developed some new and rather convincing evidence that there is a definite soil colloid. The term, "colloid", does not indicate a substance, but a state, and there are almost as many colloids as there are colloidal materials. Almost every mineral can be transformed into the colloidal state, merely by grinding it fine enough. The soil colloids represent a mixture of different colloids that are means of different materials in a very finely divided state, and the properties of these mixtures may differ from each other as widely as the properties of the soils.* Mr. M. M. McCool, of Michigan, observed a remarkable difference in color and tenacity between soil colloids that accumulated at the base and those that formed the top of the sediment in the tubes of the super-centrifuge, although both colloids came from the same soil. Some soils with 20% of colloids were found to be far more tenacious than others with 40 to 70%, on account of a difference in the character of the colloids which they contained. These few examples may be sufficient to demonstrate that the quantity of colloids present in the soil is far from determining its character. In order to learn something about the soil, it would also be necessary to determine the type of the colloids that it contains and their state of adsorbtive saturation. This, however, would require intricate chemical investigations which, for practical purposes, cannot be considered.

Hence, if all the data, ((*a*) to (*e*)), were known for two given soils, it would still be impossible to determine their similarity or differences. The difficulty is obviously in the fact, that none of the data, ((*a*) to (*e*)), has any direct bearing on the facts that interest the foundation engineer. The properties that determine the behavior of the soil in the foundation pit directly are not the uniformity, nor the mineralogical composition, nor the water content. They are:

(1) The volume change produced by an increase of the pressure acting on the soil; because part of the settlement of a building may be due to a compression of the soil. If two similar buildings are erected on equally compressible soils, they will ultimately settle the same amount.

* "The First International Soil Congress and Its Message to the Highway Engineer," by C. Terzaghi, *Public Roads*, Vol. 8, No. 5, pp. 89–94.

(2) The permeability of the soil; because the smaller the permeability of the soil the more time it takes until the excess water drains out after the construction of a foundation.

(3) The cohesion or the shearing resistance of the soil under zero load; because the cohesion determines the relation between settlement and diameter of the loaded area at equal unit pressures.*

The determination of these three soil properties is enough for practical purposes. The question concerning the causes of these properties—whether or not they are determined by colloid constituents, effective size, or mineralogical composition—belong in the laboratory.

Hence, when approaching the problem of soil classification, the writer first attempted to study each one of the practically important properties, ((1) to (3)), individually, to find out within what limits they could possibly vary.

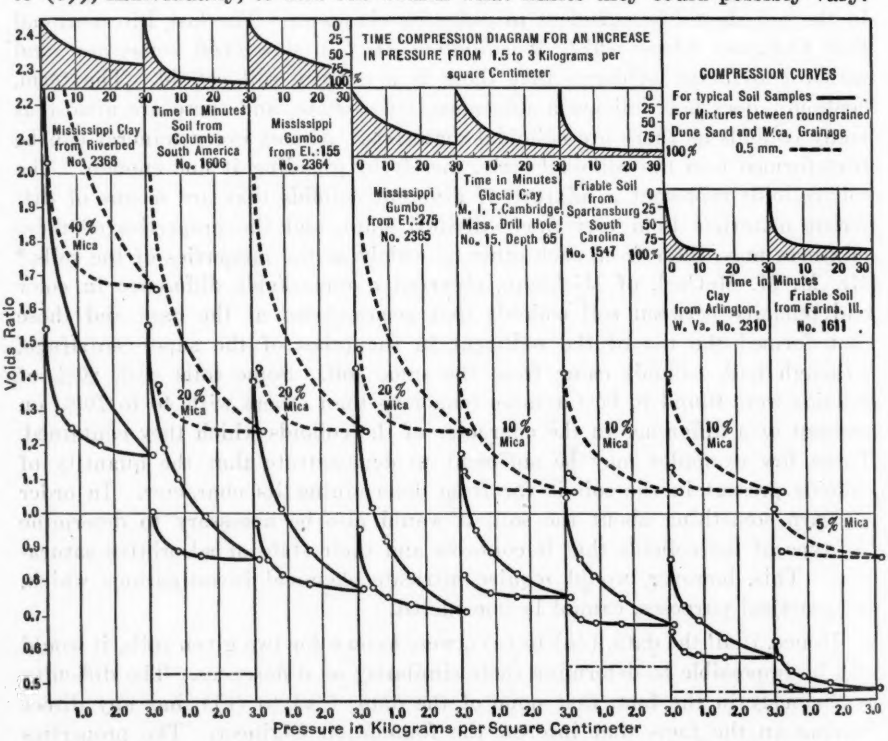

FIG. 14.—PRESSURE VOLUME DIAGRAMS FOR SAND-MICA MIXTURES AND FOR TYPICAL SOILS.

A careful study disclosed the surprising fact that compressibility of soils may vary between limits as far apart as the compressibility of concrete and rubber. To demonstrate this fact the diagrams of Fig. 14 have been plotted. The full drawn curves show the volume changes of soils, ranging between Mississippi gumbo (most compressible soil) and clean, round-grained sand (least compressible), produced by first raising the pressure from zero to 3

* "Erdbaumechanik," by Charles Terzaghi, M. Am. Soc. C. E., Franz Deuticke, Wien, 1925.

tons per sq. ft. (3 kg. per sq. cm.), and then gradually reducing the pressure to zero. At the outset the water content of each one of the samples was equal to the water content of the material after slow sedimenation in quiet water (Atterberg's lower liquid limit). From Fig. 14 it may be learned that the compressibility of the sand amounts to only a small fraction of the compressibility of the Mississippi gumbo. Even a year ago (1925) the cause of the tremendous difference in the compressibility of different soils was not yet quite clear to the writer. Suspecting that it might be due to the greater or smaller abundance of scale-like particles in the soil, he induced Mr. G. Gilboy, of the Massachusetts Institute of Technology, to investigate the elastic properties of differently proportioned mixtures of sand and mica, both with a grain size of 0.5 mm. The results of these investigations certainly were striking. By properly selecting the mica content of the sand, it was possible to imitate practically all the properties associated with the com-

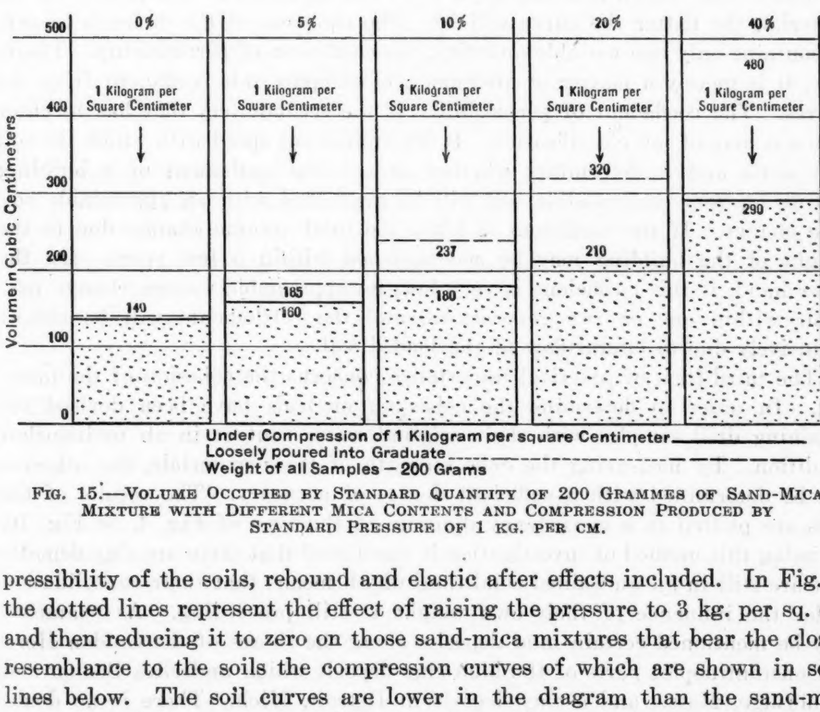

FIG. 15.—VOLUME OCCUPIED BY STANDARD QUANTITY OF 200 GRAMMES OF SAND-MICA MIXTURE WITH DIFFERENT MICA CONTENTS AND COMPRESSION PRODUCED BY STANDARD PRESSURE OF 1 KG. PER CM.

pressibility of the soils, rebound and elastic after effects included. In Fig. 14 the dotted lines represent the effect of raising the pressure to 3 kg. per sq. cm. and then reducing it to zero on those sand-mica mixtures that bear the closest resemblance to the soils the compression curves of which are shown in solid lines below. The soil curves are lower in the diagram than the sand-mica curves merely because of a difference in uniformity of the two materials. In order to make these facts clearer, the diagrams, Fig. 15, have been prepared. The upper row of numbers shows the space occupied by a standard quantity of 200 grammes of sand-mica mixtures with a mica content of 0%, 5%, 10%, 20%, and 40%, respectively. The lower row shows the compression produced by a standard pressure of 1 ton per sq. ft.

Thus, it becomes evident that one of the most important properties of the soils—compressibility—has nothing to do with the effective size, the uni-

formity, or the colloid content. It is merely the mechanical effect of a greater or smaller abundance of scale-like particles.

The property next in importance concerns the permeability of soils. The first tests that the writer made for determining the permeability of very fine-grained materials, such as clays, lasted several months.* However, a method has been devised, based on the theory of hydrodynamic stresses, by which the coefficient of permeability can be determined from not more than one dozen readings. By using the same sample that serves for investigating the compressibility, the determination can be made in 24 hours.† The test is made by first raising the pressure which acts on the sample by 100 per cent. The less permeable the material, the longer will be the time during which the excess water is squeezed out. The gradual loss of water betrays itself by a gradual settlement. By observing the settlement in specified time intervals covering a period of 24 hours, a curve of settlement against time may be obtained, as shown near the upper edge of Fig. 14. The less permeable the material, the flatter the curve will be. The equation of the curve is known. It contains only one variable quantity, the coefficient of permeability. Therefore, it is merely a matter of arithmetic to compute this coefficient from the curves. The coefficient of permeability is the second item that has to enter into a system of soil classification. It determines the speed with which the soil will settle and it determines whether or not the settlement of a building resting on very compressible soil will be associated with an appreciable volume change. If the coefficient is high, the total volume change due to the weight of the building may be accomplished within a few years. On the other hand, if this coefficient is very low, no appreciable volume change may occur within one or two generations, and the settlements will be almost exclusively due to lateral flow of the loaded soil.

The third item of practical importance concerns the cohesion of the material. In order to determine the cohesion, methods have been devised for obtaining drill samples with their original water content in an undisturbed condition. By measuring the cube strength of these materials, the cohesion can be determined with a sufficient degree of accuracy. The results of the tests are plotted in a consistency diagram of the type of Fig. 4, or Fig. 16. By using this method of investigation it was found that there are clay deposits that are still in an undrained condition, which means that their consolidation under the influence of their own weight is still proceeding. As examples, may be mentioned certain mud deposits along the shores of the Golden Horn in Constantinople; parts of the blue clay deposit which underlies Boston and Cambridge, Mass.; and a clay deposit in Detroit, Mich. There is no doubt that many others of a similar kind exist. The surface of such deposits would gradually subside even if no buildings were erected on them, and the construction of buildings accelerates the process, involving considerable settlements regardless of the type of foundation.

* "Die physikalischen Grundlagen des Technisch-geologischen Gutachtens," by Charles Terzaghi, M. Am. Soc. C. E., *Zeitschrift* des Oesterreichischen Ingenieur- and Architekten Vereins, September, 1921.

† "Die Berechnung der Durchlässigkeitsziffer der Tone aus dem Verlauf der hydrodynamischen Spannungserscheinungen," *Sitzber.* der Akad. der Wiss. in Wien, Math. Natur., Abt. IIa, 1923.

The first two items, compressibility and permeability, determine the type of soil with a greater precision than the concrete term, "1 : 2 : 4", describes the property of artificial stones. A third item, cohesion (shearing resistance) for clays, and density for sands and quicksands, determines the state in which the soil occurs. The methods for measuring the density (firmness) of sandy materials have not yet been developed, but there seems to be no insurmountable obstacle to the solution of the problems involved.

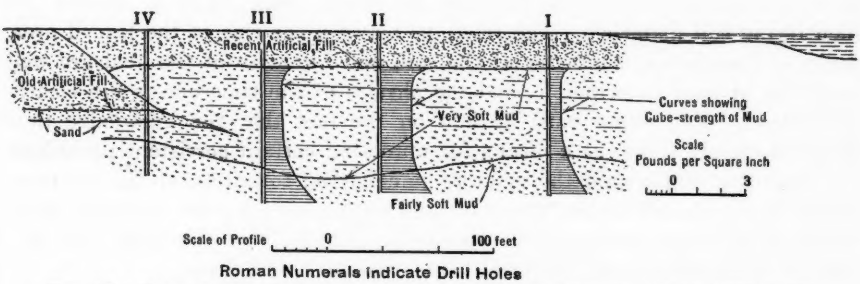

FIG. 16.—CONSISTENCY DIAGRAM FOR A NEW MUD DEPOSIT AT AÏWAN SERAÏ NEAR CONSTANTINOPLE, TURKEY.

With these three items as a basis, a soil classification could be devised which would serve to bring soils into as rational a system as that which covers artificial construction materials, because soils with equal compressibility, equal permeability, and occurring in the same state, will behave, under load, in an identical manner, regardless of what the ultimate causes of their properties may be.

However, in practice, there are two serious difficulties associated with any attempt to classify soils according to the method proposed in the preceding discussion. The first is the difficulty of obtaining undisturbed samples of certain soils. According to the results obtained by Swedish investigators, disturbing a soil without changing its volume of voids and its water content may cause the cohesion (consistency) of the soil to be reduced by an amount ranging between 75 and 96% of its original value. Hence, the conclusions derived from testing the consistency of a drill sample may be very misleading unless one has succeeded in securing the sample in an undisturbed state.

The second difficulty is due to the fact that the character of the natural ground varies to a greater or less extent from foot to foot, in a horizontal and in a vertical direction, the variations depending on how the soil deposit was formed. In order to get an accurate conception of the average character of the soil, a considerable number of samples ought to be tested. The tests required for determining the constants mentioned in the preceding discussion are too expensive, in time and money, to be performed on every individual sample. Hence, efforts are made to determine certain simple routine tests, the results of which would furnish the approximate information desired at a reasonable amount of time and labor. At present, these efforts are being carried on by the U. S. Bureau of Public Roads, in co-operation with the writer.

Conclusions

Considering the present state of knowledge in the field of soil mechanics, the prospects concerning the future of foundation engineering as an applied science are decidedly encouraging, the principal obstacles against progress in this field having been removed. The elastic properties of the most troublesome soils, clays included, are now at least as thoroughly known as those of concrete or steel. The methods of soil classification and soil identification have reached a point where it can be clearly seen what is needed for identifying soil materials. The relations between the size of the loaded area, depth of foundation, character of the soil, and intensity of the load are well analyzed, at least in principle, and the physics of the time effects are clearly understood. Yet, stress must be laid on the fact that the knowledge thus obtained merely serves as a means for uprooting certain persistent prejudices (for instance, concerning the value of pile-driving formulas, or the interpretation of the results of loading tests) and establishing a more accurate interpretation of actual construction experience. The bulk of the work—the systematic accumulation of empirical data—remains to be done.

Foundation problems, throughout, are of such character that a strictly theoretical mathematical treatment will always be impossible. The only way to handle them efficiently consists in finding out, first, what has happened on preceding jobs of a similar character; next, the kind of soil on which the operations were performed; and, finally, why the operations have led to certain results. By systematically accumulating such knowledge, the empirical data being well defined by the results of adequate soil investigations, foundation engineering could be developed into a semi-empirical science, comparable in its character to certain branches of medicine. For the first time a large-scale effort of this kind was made by the Swedish Geotechnical Committee, in its epochal investigation of Swedish landslides, 1914-22.* In connection with its work, the Committee has tested a vast number of soil samples, extracted from more than 10 000 drill holes, at different depths below the surface. On account of the time (five years) that has elapsed since the Committee finished its work, some of the experimental methods have been superseded. Nevertheless, the work of the Committee could serve as a noteworthy example for the way in which a similar enterprise should be organized in the field of plain foundation engineering.

At present, the data derived from previous experience in foundation engineering are of as little value as were those derived from Swedish landslide experiences prior to the work of the Geotechnical Committee. The information concerning soil character is inadequate, and the interpretation of the observed facts is very often arbitrary and inconsistent with the laws of physics and mechanics. Hence, the first requirement for improving conditions consists in standardizing the methods of soil classification, and the second, in consistently applying present knowledge of soil mechanics to observations in the field.

* Statens Järnvägars Geotekniska Kommission, 1914–22, Stockholm, May, 1922.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

THE CONOWINGO HYDRO-ELECTRIC DEVELOPMENT ON THE SUSQUEHANNA RIVER*

BY ALEXANDER WILSON,† 3D, ESQ.

For more than forty years the possibilities of power development in the lower course of the Susquehanna River have engaged the attention of engineers and capitalists. The first evidence of contemplated power development in this river is found in the Act of the Legislature of the State of Maryland which was passed in 1884, authorizing The Susquehanna Water Power and Paper Company of Hartford County to acquire, by condemnation, certain property necessary for the proper extension and development of its existing dam or any dam it might locate or build near that site.

This water-power development consisted of a small wing-dam in the bed of the river, an intake channel, head-gate, power plant, and tail-race. Part of this work is still in existence and comprises what might be termed the first water-power development of the Susquehanna River.

Until, however, the growth of the steam-generated electric systems in near-by cities had developed a market capable of absorbing a large part of the energy in the flow of this river, its power development was not economically feasible. In 1910, the Pennsylvania Water and Power Company completed a dam and power house at Holtwood, Pa., which now has an installed capacity of 150 000 h.p.

Early in March, 1926, The Philadelphia Electric Company, through its subsidiary, Philadelphia Electric Power Company, and the latter's Maryland subsidiary, The Susquehanna Power Company, under Federal license approved February 20, 1926, started the construction of the Conowingo Dam and Power House, with an initial wheel capacity of 378 000 h.p. (594 000 h.p. ultimate capacity), situated about 2 miles below the highway bridge at Conowingo, Md. The energy generated in this plant will be transmitted to Philadelphia,

* Presented at the meeting of the Power Division, Philadelphia, Pa., October 6, 1926. Written discussion of this paper will be closed in March, 1928. For discussions covering other phases of this project, see p. 2305 *et seq.*

† Constr. Engr., Philadelphia Elec. Co., Philadelphia, Pa.

Pa., at 220 000 volts to be utilized in The Philadelphia Electric Company's System.

As stated the dam and power house are in Maryland, but the upper half of the reservoir and the greater part of the transmission lines will be in Pennsylvania. This required joint action by the Public Service Commissions of the two States. Also, as the War Department has ruled that the Susquehanna is a navigable river, a license from the Federal Power Commission was required.

Corporate Structure

As The Philadelphia Electric Company will use practically the entire output of the Conowingo development, it was necessary that it should control the operation of the plant. It was also essential, in order to make the securities of the project attractive to the investing public, that The Philadelphia Electric Company should guarantee the completion of the project and be responsible for the payments on which the securities of the project depend for support. As The Philadelphia Electric Company is not allowed by its charter to do business in Maryland, these requirements were met by arranging to have three subsidiary corporations, as follows:

The Susquehanna Power Company, incorporated in Maryland, will own all the physical property of the project in that State, comprising the dam, power house, and tail-race, and parts of the reservoir and transmission lines.

Philadelphia Electric Power Company, incorporated in Pennsylvania, will own all the physical property in Pennsylvania, this being principally lands for the reservoir, and also the greater part of the transmission lines. This Company also owns all the stock of The Susquehanna Power Company. All the voting stock of Philadelphia Electric Power Company is owned by The Philadelphia Electric Company, which also leases the transmission lines owned by Philadelphia Electric Power Company.

The Susquehanna Electric Company, which was formed for the purpose of leasing, for the term of the license, the properties of The Susquehanna Power Company in Maryland, under contract with The Philadelphia Electric Company, will operate the plant and will sell all energy generated to The Philadelphia Electric Company.

As set up, therefore, the Conowingo hydro-electric development is being made by and for The Philadelphia Electric Company, and when completed will be operated as a part of that Company's System. The construction of the development is in charge of the Engineering Department of The Philadelphia Electric Company.

Construction Organization

The contract for the design and construction of the dam and power house has been awarded to Stone and Webster, Incorporated, which has sub-contracted the construction of the greater part of the dam to The Arundel Corporation of Baltimore, Md. This Corporation has also been awarded the contract for the relocation of the tracks of the Columbia and Port Deposit Branch of The Pennsylvania Railroad. The contracts for the design and

construction of the transmission lines, and for the switching station on the roof of the power house has been awarded to Day and Zimmerman, Incorporated. Construction progress schedules have been carefully worked out and it is expected that the initial installation will be completed so that power from Conowingo will be available to help carry the peak load of The Philadelphia Electric System in December, 1928.

Precipitation and Run-Off

With the exception of the St. Lawrence, the Susquehanna River Basin is the largest and most important on the Atlantic Coast, and embraces a total area of 27 400 sq. miles, which comprises 47% of the area of the State of Pennsylvania, 13% of the area of the State of New York, and 2% of the area of the State of Maryland.

The annual precipitation over this area, according to the records of the U. S. Weather Bureau, varies from 31.4 to 44.3 in., with a mean of 39.4 in. The run-off which eventually finds its way to the sea through the Susquehanna River, varies from 16.6 to 29.1 in. and averages 55% of the rainfall. The run-off is a minimum in August, September, and October, during which months it ranges from 5 to 30% of the rainfall and averages about 15 per cent. Like all other Pennsylvania streams, the Susquehanna River has a natural run-off which is extremely variable, both from day to day, or week to week, and from season to season.

High water frequently occurs in January from melted snow. Floods accompanied by ice gorges occur usually in March and result in a high-water level, although with a lesser volume of flow than at times of clear-water floods caused by heavy rainfalls, occurring over the whole or a part of the watershed as late as June. In the late summer or fall, periods of low water are frequently noted.

During the past century there have been several great floods in this river, the most notable of which was that of June, 1889, which was coincident with, although not caused by, the Johnstown Flood, and which probably exceeded any flood that ever occurred in this stream. It is estimated that during this flood the flow reached a maximum of 730 000 cu. ft. per sec. Reliable official records on the flow of the Susquehanna River taken for the last 31 years at Harrisburg, Pa., show that the lowest minimum discharge there was 2 200 cu. ft. per sec. This occurred in 1909.

The head-waters of this river system are on the elevated plateau which separates the waters which flow south and east into the Atlantic streams from those flowing north and west into the Mississippi, St. Lawrence, and the Great Lakes.

From Northumberland, Pa., situated at the junction of its East and West Branches, the river has generally a fairly uniform grade of 2½ ft. per mile, except for local rapids between the Muncy Dam and Conewago Falls, near York Haven. Below, or in the lower 40 miles of its course, the slope increases to an average of 5 ft. per mile to tide-water and the width of the river becomes contracted, narrowing into a gorge which, in places, is reduced to a width of 0.2 to 0.5 mile. In the last 27 miles the river drops from an

elevation of 225 ft., with an average slope of 5.6 ft. per mile, causing a swift current which has worn a low-water channel of great depth in many places.

Along this lower section, the river has cut its way through a range of table-land, and its bed is walled by steep rocky bluffs on both sides, affording excellent foundation conditions for water-power developments. A part of the fall in this section, as noted previously, has been developed by the Pennsylvania Water and Power Company's Dam at Holtwood. The Conowingo Dam will develop the head from the Holtwood tail-race to within 4 miles of tide-water.

Maximum Elevation of Pool

By agreement with the Pennsylvania Water and Power Company, a pool elevation for the Conowingo Dam, 108.5 ft. above mean sea level, has been adopted. At this elevation the water will be backed up over a part of the tail-race of the Holtwood Plant, which has not yet been excavated but which, when excavated, would have resulted in the development of increased head and power at the Holtwood Plant. It was mutually agreed that this additional head could be developed more economically at Conowingo, and it was arranged therefore, subject to approval of commissions having jurisdiction, that the Conowingo Pool shall be maintained at Elevation 108.5, the Holtwood Company to share in the Conowingo Plant's gain accruing from the increased head.

Selection of Dam Site

With the pool elevation decided upon, it was desirable, in order to develop the maximum head, to locate the dam as close to tide-water as possible. Here, however, the relocation of the tracks of the Columbia and Port Deposit Branch of the Pennsylvania Railroad Company, on the east bank of the river, imposed a limitation, in that it was necessary to provide a satisfactory grade from the present roadbed at the north end of Port Deposit to the elevation of the relocated tracks above the Conowingo Dam. A grade of 0.35% was finally accepted by the Railroad Company.

Five possible sites for the Conowingo Dam were examined and an exhaustive study was made of the head and power available and the total cost of development at each. The site as finally adopted is approximately 2 miles below the Village of Conowingo, and is far enough north of Port Deposit, Md., to permit the use of the accepted maximum run-off grade on the railroad. A reservoir having an area of approximately 14 sq. miles will be formed. The hills on either side of the river at this location form natural abutments, that on the Cecil County, or east, side rising to an elevation of 250 ft. above sea level, and that on the Hartford County, or west, side rising to an elevation of 155 ft.

The river bed and the banks, to a height well above the pond level, are of granitic formation. During the autumn of 1924 twenty-six core borings were made along the line of the up-stream face of the dam between the two abutments. These were drilled to depths varying from 5 to 30 ft. below the rock ledge, the average depth being about 15 ft. All these cores showed firm hard

granite or gabbro. In addition, four borings were made to a depth of 100 ft. below the rock ledge, and these also showed hard rock for their entire length. This rock, when crushed to suitable sizes, is excellent material for concrete. On the east bank there is ample space for the erection of the construction camp and plant and for the storage of materials. Transportation facilities are supplied by the Columbia and Port Deposit Railroad.

The main channel of the river at this site is along the west bank. The power house, therefore, is being built at this end of the dam. Space for the construction plant on this side of the river is somewhat limited, but an old canal which formerly operated on this bank has been partly filled to provide space. To supply transportation facilities for the power-house construction, it was necessary to build approximately 10 miles of railroad to connect with the Pennsylvania Railroad System at Havre de Grace, Md. The tow-path of the old Tidewater Canal afforded an excellent roadbed, requiring very little grading, and within three months of the start of construction work this railroad was in operation.

Dam and Power House

The west abutment of the dam is in a projecting hill of rock, into which the retaining wall section, extending 140 ft. from the power house to the abutment, will be built to form an adequate seal for the impounded waters. The headworks for the power house, providing for eleven main units and two station service units, then extend 950 ft. to the beginning of the spillway section, which is 2 385 ft. long. From the east end of the spillway the retaining wall section continues 1 200 ft. to the east abutment, which also serves as the abutment for the highway bridge over the relocated tracks of the Columbia and Port Deposit Railroad. The seal at this point will be made by a core-wall extending up stream into a heavy embankment at the side of the railroad. As a further protection to the railroad, a core-wall will also be extended from rock to the underside of the ties, between the abutment and the east bridge pier.

The dam is solid masonry construction of the gravity type, founded on rock, at an average elevation 15 ft. above sea level. The roadway of the highway bridge on the dam, will be at Elevation + 114, with parapet walls 5 ft. high on each side. The width at the bottom of the retaining wall section is 67 ft., and at the spillway section, 85 ft. The spillway section, designed to take care of floods up to 880 000 cu. ft. per sec., will have a fixed crest at Elevation + 86, on which will be mounted, for the purpose of regulating the level of the storage reservoir, fifty movable crest gates. These gates will be constructed of structural steel and will be equipped with two roller trains on each gate and with sealing provisions along the vertical edges and bottom. They will be 22½ ft. high by 40 ft. long, and will weigh about 42 tons each. There will also be three regulating gates, 10 by 40 ft., of similar construction, adjacent to the power house. The operation of these gates will be effected by means of three electrically operated, traveling gantry cranes, runway for which has been provided on the top of the dam and power house. Each of these cranes has been designed for a capacity of 60 tons. Each crane will be equipped with two main hoists of 30 tons each, mounted on a single trolley and operated by the same motor.

The hoisting speed will be 10 ft. per min. and the gantry travel speed, 300 ft. per min. A generator driven by a gasoline engine will be installed on one crane for emergency use.

To insure water-tightness of the foundation, grout holes are being provided on 10-ft. centers on the up-stream face. Contraction joints in the dam will be sealed with copper sheets bent Z-shaped, the top and bottom of the Z being built into the abutting sections of the dam.

At the bottom of both the retaining wall and spillway sections, a continuous drain, 10½ ft. from the up-stream face, is to be installed with transverse drains leading to the down-stream face every 45 ft. Piers for the support of the highway bridge and crest gates will be constructed on 45-ft. centers and will be 7 ft. wide.

The power house, as noted, is adjacent to the west shore of the river. The head-works provide intakes for eleven main units and two station service units. By locating the top of the intake openings 40 ft. below the pool level, protection from floating ice has been secured without the expense of constructing the usual rock-fill and skimmer arch to protect the forebay. Structural steel trash racks, in sections, 14½ by 24 ft., will be provided for all units, including the station service units. Sectional head-gates, 14½ by 25 ft., complete with bronze sealing strips, will be provided back of the trash racks for use in case it is desired to unwater the intake passages. The sectional head-gates and trash racks will be handled by the gantry traveler on the dam.

The deepest excavation for the power house will be to Elevation — 20, required for the construction of the draft-tubes for the Allis-Chalmers water-wheels. The superstructure will be of concrete, with a structural steel frame. The high-tension switching station will be on the roof.

Power-House Equipment

Seven main water-wheel units will be installed, four to be supplied by the Allis-Chalmers Manufacturing Company and three by the William Cramp and Sons Ship and Engine Building Company. Each unit will be designed to have a capacity of 54 000 h.p. at full gate, at 89 ft. normal net head, and 81.8 rev. per min., and will have a capacity of 50 000 h.p. at the point of best efficiency. They will be of the vertical-shaft, single-runner, Francis, reaction type, with plate-steel scroll case and governors of the actuator type, and will be the largest units, in physical dimensions, built to date. The runners will be of cast steel, cast in not more than four parts. Each unit will have a structural steel pit liner or generator support capable of carrying the weight of a 40 000-kv-a. generator.

The governors will have gear-driven flyballs and will be operated by oil pressure. There will be one oil-pressure system for each pair of units. An oil storage and purification system will be provided for all the governors and bearing oil for the seven main units and two station service units.

Two station service water-wheel units, supplied by the S. Morgan Smith Company, will be provided, of 1 900 h.p. each, operating at 89-ft. normal net head and 360 rev. per min. The runners will be of the single, Francis, reaction

type, equipped with vertical shafts, plate-steel scroll cases, governors, oil-pressure systems and appurtenances.

With each main water-wheel unit, a vertical-shaft, butterfly valve, 27 ft. in diameter, complete with operating equipment, oil-pressure system, and accessories, and capable of operating under an 84.5-ft. head on the center line of the valve, is to be installed in the water passages to the runners. Each valve will have a cast-steel housing and a wicket of cast steel, a forged steel shaft, and a plate-steel penstock liner extending 12 ft. up stream. These valves will be operated by oil pressure from a central oil-pressure system.

Two butterfly valves, 6 ft. in diameter, will be provided for the station service units. They will have horizontal, forged steel shafts, cast-steel wickets, cast-iron housings, and plate-steel penstocks connecting the butterfly valves to the scroll cases of the units. A motor-driven operating mechanism will be provided, capable of operating each valve under a normal and maximum head at the center of the valve of 88.5 ft. The butterfly valves will be furnished by the same manufacturers as the respective water-wheel units.

Seven main generators and two station service generators will be installed; four main generators to be provided by the General Electric Company and three main generators and the station service generators by the Westinghouse Electric and Manufacturing Company. Each main generator will have a capacity of 40 000 kv-a., generating at 13 800 volts, 3 phase, 60 cycle, and 81.8 rev. per min., will be direct-connected to a main water-wheel unit, and will be provided with a direct-connected, 715-kv-a., auxiliary, alternating-current generator and a small direct-connected exciter for the auxiliary generator. A separate motor generator set will supply the excitation current for the main generator.

The weight of the rotating elements, together with the hydraulic thrust, will be carried by thrust-bearings on top of the generators. In the generators provided by the General Electric Company a thrust-bearing of the spring type will be used, while the Westinghouse generators will have Kingsbury thrust-bearings. The guaranteed efficiency of these main generators will be in excess of 97.0% at rated capacity.

Each station service generator will have a capacity of 1 600 kv-a., generating at 460 volts, 3 phase, 60 cycles, and 360 rev. per min., will be direct-connected to one of the station service water-wheel units, and will be provided with a direct-connected, direct-current exciter and Kingsbury thrust-bearing.

Stationary power transformers will consist of four banks of three transformers, each having the following rating: 26 667-kv-a. rated capacity, 29 333-kv-a. overload rating, single-phase, water-cooled, 13 800 low-tension voltage, and 127 000/220 000 volts Y at the high-voltage terminals. The guaranteed efficiency at rated capacity is in excess of 99 per cent.

Two power station cranes will be installed in the generator room. They will be of the 4-motor, electric traveling type, each equipped with a 150-ton main hoist and a 25-ton auxiliary hoist. Both the main and the auxiliary hoists will be installed on a single trolley. The hoisting speed will be 6 ft. per min. and 30 ft. per min., respectively. These hoists will be equipped with alternating-current motors, operating at 440 volts, 3 phase, and 60 cycles.

Two 50-ton transformer traveling cranes will be installed for separate or combined operation. They will be of the 3-motor type, having a hoisting speed of 10 ft. per min. The motors will use alternating-current power at 440 volts, 3 phase, and 60 cycles.

One 15-ton machine-shop traveling crane of the 3-motor type, equipped for control from the floor, will be installed. The electrical characteristics of the motors will be similar to those of the other cranes. A small gantry crane will be installed on a runway over the stop-log slots on the down-stream side of the station, to be used for handling the stop-logs.

The tail-race for the seven main units and the two station service units of the initial development, will require the excavation of approximately 400 000 cu. yd. of rock, of which it is expected a considerable part will be crushed and used as concrete aggregate.

Progress Schedule

The progress schedule contemplates the completion of the initial project in two low-water seasons. Construction camps have been established on each side of the river, that of the The Arundel Corporation, which is constructing the greater part of the dam, being on the east side, and that of Stone and Webster on the west side.

During the low-water season of 1926, a coffer-dam, enclosing the power house and west branch of the tail-race, was constructed, to a height such that it will not be topped by any flood of less volume than 400 000 cu. ft. per sec., so that work on the heavy excavation within the enclosed area can be continued through the spring floods of 1927; also, a coffer-dam has been constructed from the east bank of the river out to within approximately 700 ft. of the power-house coffer-dam. The schedule requires that the dam foundations within the area enclosed by this coffer-dam be poured up to Elevation 22 with alternate sections of spillway up to Elevation 30, or higher, by January 1, 1927. The latter coffer-dam has been built with stop-logs alternating with the rock-filled cribs, so that as soon as the alternate sections are poured to Elevation 40, the stop-logs can be removed, to allow additional passage for the spring floods.

During the low-water season of 1927, the coffer-dam from the east bank will be continued over to the power house, the river flow being taken care of through the stop-log openings in the 1926 coffer-dam and the alternate sections of spillway at Elevation 22. As soon as the foundation of the dam can be poured in this coffer-dam, with alternate sections to Elevation 40, the stop-logs in the coffer will likewise be removed.

As the piers for the highway bridge will be included in the alternate sections of the spillway, which are to be carried up ahead of the closure sections, these must be pushed to completion so that the highway bridge on the dam can be opened to traffic, and thus permit the removal of the existing highway bridge at Conowingo.

By the close of the low-water season of 1927, the head-works of the power house should be completed. The relocation of the tracks of the Columbia

and Port Deposit Railroad must also be completed and in operation by this date. In the fall of 1927, closure of the spillway sections will be commenced and is scheduled to be completed before the spring floods of 1928. During final closure of the spillway, it is planned to pass the river flow through light 10 by 18-ft. sluices built through the dam. These will be closed finally by means of specially designed gates.

Construction work was started on March 8, 1926, and excellent progress has been made on the work both sides of the river. The work of relocating the tracks of the Columbia and Port Deposit Branch, was commenced on May 25, 1926, and is also making very satisfactory progress.

The construction of the dam and power house will require approximately 650 000 cu. yd. of concrete, of which, 400 000 cu. yd. will go into the dam. For the construction of the dam, a steel construction bridge with a deck at Elevation 60, is being built on the toe of the apron. On this bridge are three self-propelled, electrically-operated, gantry travelers, each equipped with a 15-ton derrick and a concrete hoist tower. There are also three standard-gauge railroad tracks on the bridge, and the gantry cranes are constructed so that cars on all three tracks can pass under them. The concrete-mixing plant on this side of the river is designed so that the concrete is discharged from the mixers directly into cars at the bridge elevation. Gasoline locomotives are being used to transport the concrete. Practically no rock excavation for the dam foundation has been required, it having been found necessary only to clean off the over-burden, remove the loose rock, and clean out the joints in the ledge rock.

For the construction of the power house and tail-race, the excavation of approximately 360 000 cu. yd. of rock is necessary, part of which will be crushed and used as concrete aggregate. A suitable crushing plant has been installed, with facilities for storing the crushed rock. In the placing of concrete for the power house, it is planned to use six steel towers, 250 ft. high, with 16-in. chutes.

A total force of approximately 3 000 men is now engaged on this work. It is expected that the first unit will be ready for operation about the middle of 1928, and that several additional units will be ready to help carry the peak load in Philadelphia that year.

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PAPERS AND DISCUSSIONS

THE CONOWINGO HYDRO-ELECTRIC DEVELOPMENT ON THE SUSQUEHANNA RIVER

Discussion*

By Messrs. William C. L. Eglin, J. V. Hogan, and A. W. Clark.

William C. L. Eglin,† Esq.—Any one who has never been intimately connected with a project of this nature, cannot appreciate the amount of preliminary work that must be done before actual construction can be commenced. If the preparatory work is not done carefully and thoroughly, disaster may overtake the project before it is completed.

In this project there was an enormous amount of preliminary engineering investigation and study required. Many problems of legal procedure and authorization, involving the Federal Power Commission, the Public Service Commissions of both Maryland and Pennsylvania, State Highway Commissions, County Commissions, Water Boards, Sanitary Boards, and numerous others, had to be satisfactorily disposed of before the project could be launched.

Formerly, an Act of Congress was required before such a development could be made, but with the enactment of the Federal Water Power Act in 1920 a Federal Power Commission was created, having jurisdiction over all hydro-electric developments in navigable streams. It has always been contended that the lower course of the Susquehanna River was not a navigable stream but, as the Judge Advocate of the War Department decided that it was, it was necessary to secure a Federal license.

It has been pointed out that the dam and power house (the greater part of the investment) is located in Maryland, but that the pool extends into Pennsylvania. This made it necessary to secure the joint approval of the

* This discussion (of the paper by Alexander Wilson, 3d, Esq., presented at the meeting of the Power Division, Philadelphia, Pa., October 6, 1926, and published in this number of *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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Public Service Commissions of both States. Under the Federal Act the Power Commission takes jurisdiction in all matters upon which the State Commissions are not empowered to rule. In the Conowingo Project, neither of the State Commissions could approve the issuance of securities, and it devolved, therefore, on the Federal Power Commission to make its first official finding in this connection.

When The Philadelphia Electric Company became interested, the first work was to examine all the reports that had previously been made on the development of the power resources of the Susquehanna River. The most capable engineers of the country did some real heroic work in earlier days studying this problem. The late Alfred Noble, Past-President, Am. Soc. C. E., prepared one of the most exhaustive reports on the possibilities of the hydro-electric development on the river. In fact, that report was used as a basis for the final investigations. In the preliminary work, five dam sites, pointed out by Mr. Noble were reconsidered. The final site is not the one recommended in his or in any of the earlier reports. It is known as Site 4A, that is, it is a new site, south of Mr. Noble's Site No. 4.

The rapid advance that has taken place, not only in hydro-electric development and hydro-electric machinery, but in the demand for power, changes all the conclusions of the earlier reports, and those that are even a year or two years old are practically out of date to-day. The first one that was made for The Philadelphia Electric Company, contains approximately 230 pages and 50 or 60 plates.

It includes a study of: The water-shed, precipitation, run-off, and the Susquehanna River area flood; ice formation; the quantity of water available for water power development (from official records 1901 to 1927) and the effective head of water that can be secured; the investigation of various sites and locations for hydro-electric plants; the comparison of cost of hydro-electric development at each site; recommendations as to the selection of the most desirable site and a plan for the ultimate development of the total head available; a final analysis and review of the various alternatives possible for the development of Site No. 1 from the economical standpoint, utilizing a maximum of 50 000 cu. ft. per sec. of river discharge; a selection of sites, comparison of costs and possible development; transmission lines and sub-stations, comparing cost of the price of line, line losses, and operating costs; the estimated gross and net output and power deliverable from the hydro-electric plant (operating under probable average conditions) to the sub-station at the end of the transmission line; a complete detail of the operating expenses of the hydro-electric development at each site; a general summary of the project, its operation and operating expenses, utilization, number of units, distribution of loads between the hydro-electric and the steam plant for 1925 to 1928; total capital requirements for the development, including working capital and miscellaneous expenses; forecast of earnings and expenses for the first year of operation and estimates of increase in net earnings from year to year during the first three years of operation; a brief history of the Susquehanna Power Company and other corporations, consolidated or related to

this project; opinions from counsel regarding the status of the development; and the approximate requirements of market within reasonable transmission distance.

After this report was digested the next step was to apply for preliminary permits from the Federal Power Commission and when that application was made, all the material was discussed with the Federal Power authorities, then with the Maryland Public Service Commission, and, finally, with the State of Pennsylvania. As stated, innumerable hearings before public bodies were held. In one report, made to one commission, there were 125 pages of typewritten matter and 9 blue-prints, together with exhibits containing 150 pages and many blue-prints, there being more than 20 blue-prints and about 300 pages of typewritten reports prepared for one commission alone. This illustrates the tremendous amount of preliminary work that was necessary before construction could begin. Many of the engineering conferences started at 9:00 A. M. and ended at 11:00 P. M. The Federal Commission and the Public Service Commissions of both Maryland and Pennsylvania gave their fullest co-operation. It was a long drawn out investigation, but the immensity and importance of the problem made it thoroughly justifiable.

J. V. HOGAN,* ASSOC. M. AM. SOC. C. E.—The work of The Arundel Corporation on the Conowingo Hydro-Electric Project is divided into two contracts. The first is for the construction of the dam from its east end to a point about 50 ft. east of the power house, a length of 3 600 ft. The second contract is for the relocation of the tracks of the Columbia and Port Deposit Branch of the Pennsylvania Railroad, which will be flooded by the construction of the dam. The railroad is to be changed from Port Deposit, Md., to Fishing Creek, Pa., a distance of 16 miles.

Under the Arundel contract 1 200 ft. of the dam comprises the east retaining wall section, and the remaining 2 400 ft. is the spillway section, of which, 1 450 ft. is in the river and must be built in a coffer-dam. The Pennsylvania Railroad tracks parallel the river about 1 100 ft. from the east bank, and the level area on the down-stream side of the dam, between the railroad and the river, affords space for railroad sidings, storage yards, shops, and construction plant. The camp and office buildings are on a narrower level strip between the railroad and the hillside.

Work was started on the dam on March 8, 1926, and the erection of quarters for workmen was begun immediately. The camp has bunk houses, each 20 by 64 ft., built in pairs, connected by a room with showers, lavatories, and flush toilets. Each pair of houses accommodates 56 men in single-deck bunks. The office men are housed in a two-story dormitory containing thirty-six single rooms, each with lavatory and running water. The mess hall has a seating capacity of 600 men. A refrigerating plant, laundry, fumigating house, hospital, and general store serve the camp. The administration building is a two-story frame structure with hollow-tile vault.

A fully equipped machine-shop and a blacksmith shop, for handling heavy forging, have been provided; also a carpenter shop with a platform adjoining

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it for laying out large form panels. Standard-gauge yard tracks run through all shops. The general storeroom is 50 by 100 ft. Of the standard-gauge railroad tracks 5.8 miles have been laid in the yard, including 1 mile of tracks on elevated trestles.

The water supply is pumped from Octoraro Creek, a distance of 4 000 ft., to a 100 000-gal. storage tank on the hill behind the camp. The water for domestic purposes is filtered and chlorinated, while that for industrial use and for concrete is taken directly from the raw-water storage tank. Hydrants for fire protection are placed on the raw-water mains. Sanitary sewers drain from the camp buildings to two septic tanks. The effluent from the tanks is treated with chlorine before it is discharged into the river.

For the dam it is necessary to remove 170 000 cu. yd. of earth overlying for a depth of 20 ft., the rock on which the dam is to be founded. This is being done by a 2½-yd., railroad type, steam shovel, with 6-yd., standard-gauge cars and 40-ton steam locomotives. The rock when exposed was seamy and badly disintegrated on top. However, as it was desired to prevent shattering of the foundation, very little blasting could be done, except at places where it was apparent that large quantities of rock would have to be removed. Therefore, the rock is being loosened by barring and wedging and by the use of air drills and breakers. Locomotive cranes handle the rock in skips to dump cars, from which it is dumped into the coffer-dam cribs. In places, 30 ft. of faulty rock have been removed. Three motor-driven air compressors with a total capacity of 2 600 cu. ft. per min. furnish air for this work. Five 25-ton steam locomotive cranes and two gasoline crawler cranes are being used in cleaning up the bottom.

The first coffer-dam was started on April 10, 1926, and was pumped out on July 10. This coffer is 170 ft. wide inside and 1 100 ft. long. The rock-filled cribs are 16 ft. wide, with a 10-ft. opening between them spanned by stop-logs. The water side of the cribs and stop-logs is sheathed with a double layer of 2-in. plank, outside which earth from the excavation is banked. Fig. 1 is a view of the site, and the coffer-dams. The up-stream cribs are carried to Elevation 32.5 and are 30 ft. long, and the down-stream cribs, which are 24 ft. long, to Elevation 28.5. The coffer-dam was built on the rock bottom of the river, which is at an average elevation of +15, by locomotive cranes working ahead on the finished cribs. The opening between this coffer-dam and that on the west side of the river passed a flood of 120 000 sec-ft. without overtopping the cribs. The second coffer-dam was 850 ft. long and extended from the first to connect with the coffer-dam constructed for the power house, and covered all the necessary foundation work to complete this phase of the construction in the second season. The second-year coffer-dam was constructed with alternate crib and stop-log sections similar to the first-year coffer, with the exception that the up-stream coffer-dam cribs were carried to Elevation 42.5, and the down-stream cribs to Elevation 28.5. Up stream the cribs are 20 by 40 ft., and down stream, 16 by 24 ft.

For purposes of stream control during construction, the spillway was concreted in alternate sections above Elevation 20. A section, 52 ft. long, carry-

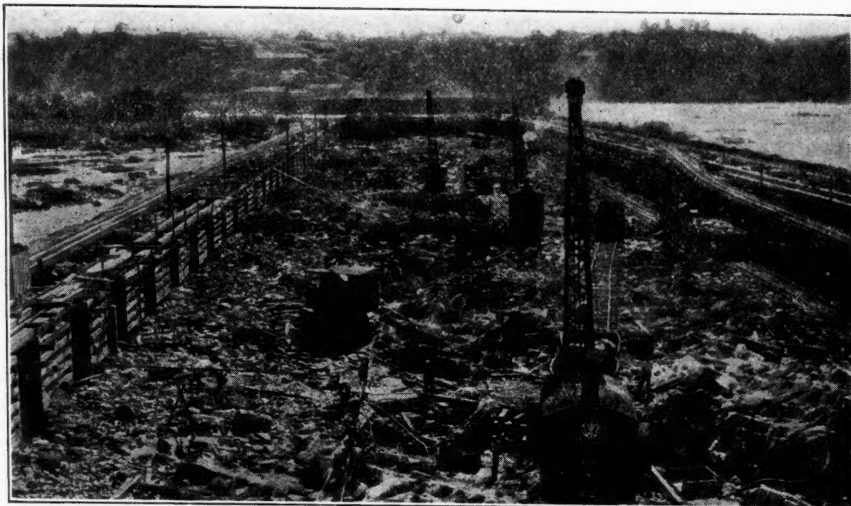

FIG. 1.—VIEW SHOWING TYPE OF CONSTRUCTION USED IN COFFER-DAM OF ALTERNATE CRIB AND STOP-LOG SECTIONS.

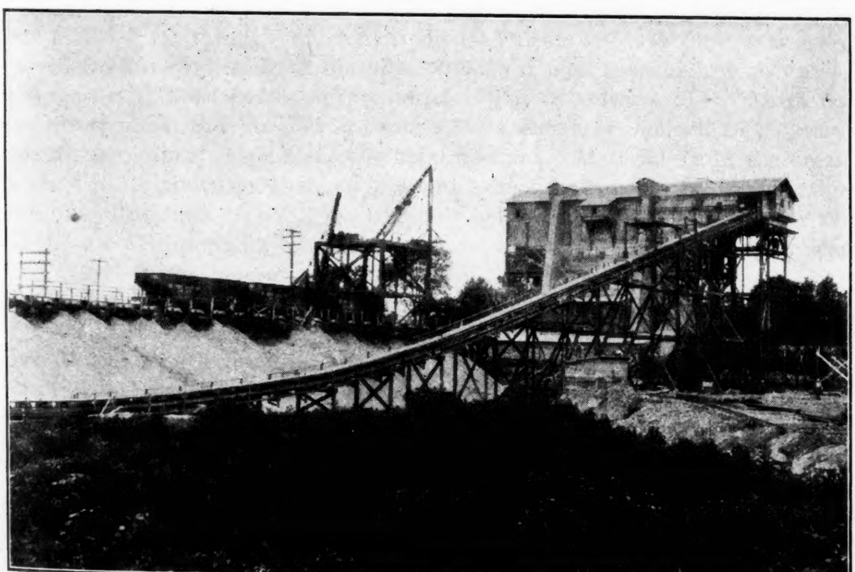

FIG. 2.—VIEW OF MIXER PLANT WITH BELT CONVEYOR, ALSO, SHOWING PART OF SAND STORAGE TRESTLE.

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ing two highway bridge piers was poured, and the adjoining section, 38 ft. long, the clear space between piers, was left open. After these alternate sections were built above high water in the first coffer, the stop-logs were removed, and the river passed through the openings while work in the second coffer-dam was being done. When all the 52-ft. sections have been built to full height, and the highway bridge is completed across the dam, the open sections will be concreted in approximately 5-ft. lifts, the river flow at that time being passed as far as possible through eight 10 by 18-ft. temporary sluiceways constructed in the dam foundation, and arranged so that when the concreting of the alternate spillway sections is completed, these can be closed by means of a drop gate and filled with concrete after the water has risen in the pool to the spillway elevation.

In this work a total of about 430 000 cu. yd. of concrete will be placed. As all the rock from the excavation is needed for weighting the cribs, gravel is being used for the coarse aggregate of the concrete. Some of the gravel is being dredged near the head of Chesapeake Bay, and the remainder shipped from plants near Baltimore, Md. The Bay gravel is towed in scows to Port Deposit, Md., the head of navigation in the Susquehanna, where a derrick loads it into railroad cars, to be hauled 4 miles to the dam site. Some crushed stone has also been purchased for concrete. The sand is obtained from the same sources as the gravel.

On account of ice in the Upper Bay during the winter, which often prevents towing, a storage of 70 000 cu. yd. of sand and gravel has been provided. This material is stored under a pile bent trestle 20 ft. high, with a total trackage of 2 800 ft., through which it is dumped from 70-ton hopper cars as received from the railroad. A bin of 1 800 cu. yd. capacity, which is also under the trestle, is filled before any material is put in storage. Five 24-in. belt conveyors under this bin deliver on to a 36-in. conveyer inclined 15°, running 450 ft. per min., which feeds the mixing plant. Material from the trestle storage is reclaimed by locomotive cranes which load into cars dumping into the bin mentioned. The 36-in. conveyor delivers on to another shorter belt at the top of the mixing plant, equipped with a tripper for distributing sand and gravel in the hoppers over the mixers.

Cement is received in bulk in box cars, and is unloaded into the boots of two bucket elevators. Screw conveyors at the heads of these elevators distribute the cement into small hoppers over the mixers. When these hoppers are full they overflow into the main cement storage bin, which has a capacity of twenty carloads. This bin, in turn, feeds back to the elevator boots when cement is being taken from it. The mixing is being done by four 2-yd., motor-driven, tilting mixers. The sand and gravel are proportioned by inundators and batchers, respectively, and the cement is weighed on automatic scales. Fig. 2 is a view of the mixer plant, with belt conveyor, and a part of the sand storage trestle.

A construction trestle of structural steel is located at the down-stream face of the dam. The top of the rail on this trestle is at Elevation 60, the roadway of the highway bridge being at Elevation 114. The bents are at pier centers in the dam, making a 45-ft. bay. Gantries operating on the trestle

have a 40-ft. span, which is the width of the trestle between the outside column centers. Three standard-gauge tracks are carried on the trestle between gantry rails. This trestle is erected on concrete piers with the top at Elevation 20, and is carried ahead as fast as the rock bottom can be prepared. Except the three main girders which carry the 45-ft. span, and which are riveted plate girders, all members of the trestle are standard rolled shapes with a minimum of fabrication, making for low first cost and high probable salvage. Fifty-four 45-ft. bays, or 2 430 ft. of this trestle, a total weight of 2 400 tons, will be erected west of the present Pennsylvania Railroad tracks. The concrete east of the tracks will be placed by a tower and chutes. Fig. 3 shows the construction bridge with gantry and movable tower.

Three self-propelled gantry travelers operate on the construction trestle. These are steel towers, each of which has a 15-ton stiff-leg derrick mounted with its mast at one up-stream corner, and a concrete hoist tower 100 ft. high near the other up-stream corner. The derricks are all steel, with 80-ft. booms and 2-drum, 100-h.p. electric hoists and attached swingers. The foot-blocks of the derricks are 45 ft. above the trestle floor, or at Elevation 105. These derricks are used for erecting forms and steel, and for setting castings; and, in case cyclopean concrete should be placed, they would be used for handling plum stones. The concrete towers receive the mixed concrete from cars on the trestle, and elevate it for delivery through chutes to the higher part of the dam. Each tower has a 30-ft. boom section of chute supporting a 30-ft. counterweight chute at its outer end. The gantries have a travel speed along the bridge of 50 ft. per min.

For the lower parts of the work, two chuting outfits suspended from, and moving along, the up-stream trestle girder, are provided for receiving concrete direct from the cars without being elevated. About 25% of total yardage of concrete can be poured through these chutes.

Concrete is hauled from the mixing plant to the towers and chutes by 8-ton gasoline locomotives geared to a speed of 16 miles per hour. Four of these locomotives are now being used, each pulling a 36-ft. standard-gauge flat-car, on which is mounted four 2-yd. side-discharge hoppers. The cars are run directly under the mixers, which are placed so that two mixers may discharge into alternate hoppers on one car simultaneously. The car is then moved up far enough to take the batches from the other two mixers in the remaining hoppers. This plant was designed to mix and place an average of 1 000 cu. yd. per day, working single shift, with a maximum output of 2 500 cu. yd. It has operated very successfully, placing 56 000 cu. yd. of concrete in 1 month, the maximum daily pour to date (October, 1927) being 3 627 cu. yd. in two shifts. A batch from the mixers actually produces 2.22 cu. yd. of concrete in the dam. Wooden forms are being used, made up in large panels which are handled by the traveling derricks. A daily force of about 800 men is employed on the dam construction, including a night shift of about 200 men.

The Columbia and Port Deposit Branch of the Pennsylvania Railroad, at present a single-track road with passing sidings, is to be revised as to alignment and grade for 16 miles. The new road-bed, however, is to be graded

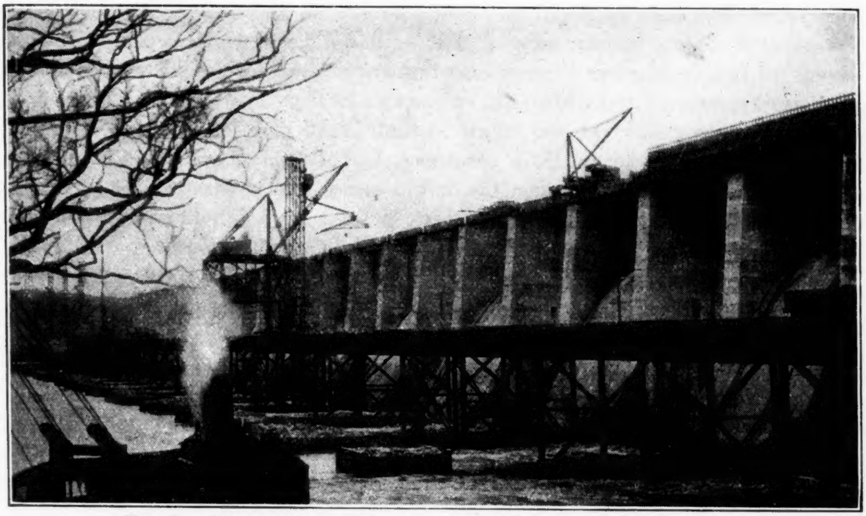

FIG. 3.—CONSTRUCTION BRIDGE WITH GANTRY AND MOVABLE TOWER.

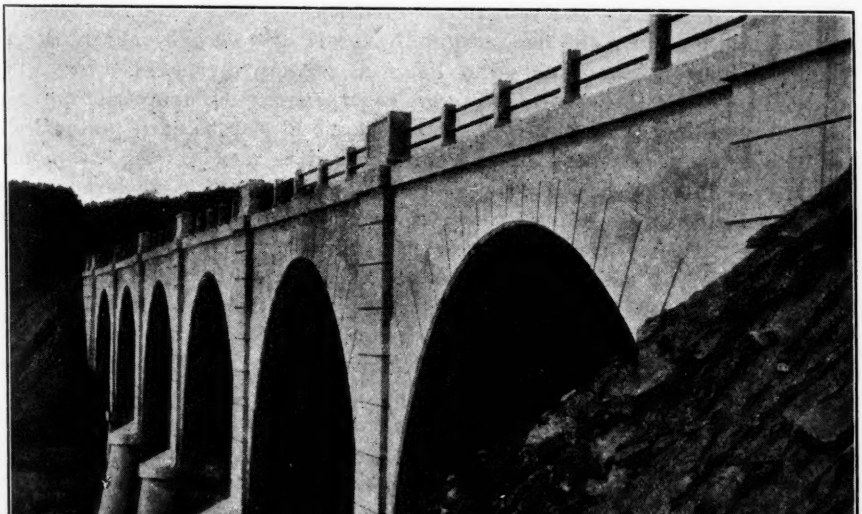

FIG. 4.—TYPE OF REINFORCED CONCRETE ARCH BRIDGE CONSTRUCTION USED ON RAILROAD RE-LOCATION.

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for double track the entire length of the change. The relocation follows very closely the 118-ft. contour on the east slope of the river valley down to the dam from where it runs off, on a — 0.35% compensated grade, 4 miles to Port Deposit. The hillside is in general very rugged and rocky, and in places so precipitous that the new line approaches the operating line very closely.

As an average of thirty heavy freight trains per day pass over the present road, and as this traffic cannot be interrupted until a single track on the new line is put in operation, it is necessary to shift the present tracks away from the toes of new fills at the points of proximity of the two lines, and to limit blasting to moderate shots practically over the entire line. It is estimated that 1 490 000 cu. yd. of grading will be required, of which, 880 000 has been obtained from cuts and the remainder will be borrowed. In the cuts it developed that 67% was solid rock.

In order to complete the double-track road-bed it became necessary to operate a single track on the relocated line and abandon the old line. This has been done, and under date of October 2, 1927, the first train was operated over the relocated line. This meant a total excavation and borrow of 1 109 000 cu. yd. in 15 months, the start having been made May 25, 1926. This time limit, coupled with the difficulties mentioned, required that the grading be attacked at a comparatively large number of points. Nine steam shovels were used on the line. Four of these are standard railroad shovels with 2½-cu. yd. dippers, one is a 2½-yd. shovel mounted on caterpillars, three are full revolving caterpillar shovels with 1¾-cu. yd. dippers, and one a full revolving caterpillar with 1¼-cu. yd. dipper. A small revolving shovel is being used for making fills where the present tracks are to be shifted.

Hauling from shovels to dumps is done over a 36-in. gauge track in 5-yd. dump cars. Four 14-ton and seven 8-ton gasoline locomotives and two 16-ton steam locomotives are being used. In several places, tractors and 5-yd. dump wagons made fills. Portable gasoline-driven compressors drive the jackhammers and tripod drills on the rock work. Nineteen compressors, each of a capacity of 309 cu. ft. per min., are used. A well drill worked in some of the deeper cuts remote from the present railroad where heavy shots could be made. Some of the rock is extremely hard and seams of quartz have been encountered at places, with consequent delays to drilling.

In addition to excavation and embankment, there are three tunnels, three arch bridges, ten arch culverts, and a number of reinforced concrete pipe culverts on the railroad work. Fig. 4, a view of Octoraro Bridge, shows the type of reinforced concrete arch bridge construction on this relocation. The three tunnels are in solid rock and have a total length of 1 020 ft., representing 26 000 cu. yd. of excavation. A battery of portable compressors furnished air for drilling. The section is semi-circular, 30 ft. wide, and 24 ft. high at the center, and was driven with a heading and two benches. Mucking was done by a ⅞-yd. gasoline, electric shovel, with the engine-generator set placed outside the tunnel.

The arch bridges are solid barrel, earth-filled, multi-span structures of reinforced concrete. These carry the road-bed over streams at Peach Bottom, Pa., and Conowingo and Octoraro, Md. (Fig. 4), and have three, five, and six

70-ft. spans, respectively. The bridges contain a total of 31 000 cu. yd. of concrete, which was placed with 1-yd. mixers and tower chuting plants. The construction of these bridges was scheduled so that a set of steel centers for three spans were used to complete the three bridges within schedule time. Seven 15-ft. arch culverts, one 20-ft., and one 30-ft. are required on the relocated line. All these, with the exception of the 30-ft. culvert, are now complete and were poured with ½-yd. portable mixers.

The track-laying was completed, using a maximum of four work trains with crews. This was very heavy work, the specifications requiring 130-lb. rail in general, and creosoted ties, together with about 2 miles of track laid with reinforced concrete ties. The ballast was either cinders or slag, and was spread direct from hopper cars. In general, the ties were hand-tamped, and the surfacing was done by track-jacks; but on the concrete ties a Nordberg track-shifter was used, and the track-tamping was done by air tools.

An average force of about 650 men was employed on the railroad relocation work, reaching a maximum of 1 400 during the track-laying period. The men were housed in small camps along the line of the relocation, and in camp cars which were shifted from point to point as occasion demanded.

A. W. Clark,* Esq.—The west side of the Susquehanna River, at the site of the Conowingo Hydro-Electric Development, is beautiful rolling farm country. The nearest railroad point, the small Town of Conowingo, is on the east side of the river, on the Columbia and Port Deposit Branch of the Pennsylvania Railroad. The river is spanned at this point by a long and very old highway bridge in poor condition, capable of sustaining a load of not more than 5 tons.

In attacking the work on the west side, it was necessary to build a railroad at the earliest possible moment for transporting materials to the site. A single-track road with an ample number of sidings was constructed from Havre de Grace, Md., on the Philadelphia, Wilmington and Baltimore Division, of the Pennsylvania Railroad, ten miles up the west side of the river, to the dam; 85-lb. rail was used throughout, and the road is ballasted with material secured from the waste bank of a stone-crushing plant, through which the railroad passes. A classification and storage yard has been built about 2 miles down stream from the work. In addition to hauling the freight necessary for construction, a few passenger trains are operated on a schedule planned to meet the working hours of employees who reside south of the dam site and in Havre de Grace. The movements of all trains are controlled by a despatcher near the works.

The first work in the field was the construction of a labor camp, started on March 9, 1926. For more than two months, until the railroad was placed in operation, it was necessary to haul all material to this site by trucks and teams.

A camp with mess halls, commissary store, and hospital has now been constructed on the high ground adjacent to the west side of the river. It is capable of taking care of the needs of approximately 1 500 men. A limited

* Works Mgr., Div. of Constr. and Eng., Stone & Webster, Inc., Havre de Grace, Md.

number of 2-room and 5-room cottages have also been built for the use of employees with families. The camp streets and all buildings are lighted with electricity, the current for both power and lighting being supplied from the Holtwood Plant of the Pennsylvania Water and Power Company, over a transmission line built from Delta, Pa., about 12 miles distant. The water supply for the entire work is secured from the river, the domestic supply being filtered, chlorinated, and pumped to a 100 000-gal. steel tank, and from there distributed through pipe lines to the camp and the work. A sewerage system is provided with the necessary septic tank, the effluent of which is chlorinated before discharging to the river.

The camp is policed by uniformed guards who are vested with the authority of deputy sheriffs, and a barred room has been set aside as a jail for those who have imbibed too freely and insist on disturbing the peace and quietude of the community.

The area of the river covered by the power plant has been divided into two sections. The section containing the head-works, power house, and a part of the tail-race was first enclosed with a coffer-dam, and while excavation was being done in this section, the construction of the coffer-dam enclosing the remainder of the tail-race could proceed. The first section was completed and unwatered during the last week of July, 1926.

The coffer-dam consists of a double row of rock-filled timber box cribs separated by a sheathed puddle-chamber approximately 10 ft. wide. The cribs were built of commercial lengths of 10 by 12-in. Southern pine and Douglas fir and were tied together across the top of the puddle chamber with the same material, securely bolted in place. The sheathing of the clay chamber was a double course of 2-by 10-in. planking, with lapped joints.

The cribs were built up in place and carefully fitted to the bed of the river, with the bottom bearing members placed lengthwise in the direction of pressure. Caterpillar cranes, that traveled on standard-gauge tracks along the top of the completed cribs, were used to unload timbers from cars. The same tracks were also used for supplying the rock-fill and material for the clay puddle which was dumped from 6-yd. and 12-yd., two-way, dump cars.

The following quantities give an idea of the magnitude of the work necessary for completing the first section of the coffer-dam which encloses the head-works, power house, and a part of the tail-race, and comprises an area of 14.7 acres: 2 575 lin. ft. of coffer-dam; 5 600 000 board ft. of timber; 69 000 cu. yd. of rock-fill; and 32 000 cu. yd. of clay.

The second section of the coffer-dam which encloses the remainder of the area covered by the tail-race, 11.1 acres, was completed the first week in June, 1927, and contains the following quantities: 2 220 lin. ft. of coffer-dam; 2 700 000 board ft. of timber; 24 000 cu. yd. of rock-fill; and 13 200 cu. yd. of clay.

In planning the excavations for the first section, it was decided to concentrate on the head-works and power house, so that they could be constructed while the excavation for the tail-race was being done. The first-mentioned work was completed the last week in April, 1927, and consisted of the removal

of approximately 100 000 cu. yd. of rock. The total quantity removed for the tail-race was approximately 200 000 cu. yd. of rock.

The excavation for the head-works and power house was carried down to several different elevations, the deepest and most difficult being in the area containing the draft-tubes. This consisted in cutting a gash in the river bed approximately 550 ft. long, 80 ft. wide, and 30 ft. deep, with the up-stream side vertical, and the down-stream side sloping up in the tail-race at a rate of 1 on 6. For this particular work, a high-lift, full-revolving steam shovel on caterpillars, with a 2.5-yd. bucket, was selected. Its use minimized the number of track changes and also made it possible to remove a large quantity of material with the hauling tracks on the bed of the river or on the top of the cut. For the remainder of the work, a 3-cu. yd. steam shovel, first operated on railroad trucks and, later, mounted on caterpillar trucks, was used where the lifts were low, and two small revolving shovels on caterpillars were used for odds and ends. All this equipment was served with 6 and 12-cu. yd., two-way dump cars, hauled by steam locomotives on a standard-gauge track system. A lead connected the construction track to double-track trestle, running parallel to the down-stream leg of the coffer-dam on a 2.5% ascending grade.

Four 800-ft. compressors located on a side hill well above the possibility of flood, supplied air for the rock drills. The rock from the lower tail-race coffer-dam was dumped on an island which divides the tail-race into two channels. The dump was made from a track on a 3% grade laid on trestle and fill.

Part of the rock in the excavations was suitable for concrete and was crushed and used. A crushing plant was installed on the west bank about 1 500 ft. down stream from the dam. Stone was carried from the tertiary crushers on a belt conveyor to a 25 000-cu. yd. storage pile under which, at sufficient intervals, there were bin gates for delivering the crushed stone to a belt conveyor operating in a 6 by 6-ft. reclamation tunnel under the entire length of the pile and then up an incline to the stone bins over the concrete mixing plant. Around the bin gates were steam coils to deliver live steam to the material in winter.

Sand was delivered to the job in standard hopper bottom cars hauled over the construction railroad from Havre de Grace, and dumped from a trestle to a storage pile under which there is a 6 by 6-ft. tunnel with conveying equipment for carrying the sand to the mixing plant similar to that used on the stone.

The concrete mixing equipment consisted of two 2-cu. yd. motor-driven mixers supplied with aggregates delivered into the mixers from batchers and inundators which were filled from overhead bins. The proper proportion of cement was determined by weight. Bulk cement was stored in wood silos by means of bucket elevators operating from the track level on the ground. Wherever it was feasible, steel supports were used in the construction of the concrete plant in order to reduce the fire hazard.

Concrete was hauled from the mixing plant by 8-ton gasoline locomotives in 2-cu. yd. standard gauge, specially designed tilting cars, of the "roll-

over" type. Four wooden towers, approximately 220 ft. high, supported the necessary chuting equipment.

For repairs to construction equipment a shop group was established about ½ mile down stream from the dam. A machine shop, blacksmith shop, pipe shop, and repair pits, equipped with an air hammer, lathes, grinders, drill presses, and other machine tools for the handling of such work, was built. As far as possible, equipment was purchased for these shops, that could be installed in the permanent shops.

A carpenter shop was constructed, in which the necessary wood-working machinery to facilitate the construction of forms, was installed. Space was reserved for lumber storage adjacent to the shop.

A place was also reserved in this vicinity for the storage of reinforcing steel, and, here, a power shear and power bender were installed for the fabrication of reinforcing steel. All these shops and the steel yard were served with standard-gauge tracks, which connected with the construction railroad and the track system throughout the work.

The power station is of structural steel and concrete. The work was arranged so that the concrete necessary to support the steel frame was placed as early as possible, in order that the structural steel could be erected and traveling cranes put in operation to handle the installation of the heavy equipment.

The draft-tube concrete was the first step in this program, and concrete was completed in the elevation required for setting the draft-tube liners during the first week in June, 1927. As this work progressed, it was followed closely by concrete walls, carried to the main floor level, which allowed the erection of structural steel to start in the early part of June, and permitted the operation of the main building crane by the end of July.

By making use of the five 10-ton derricks which served the concrete operation, the erection of the water-wheel equipment was started during the latter part of April. In this way, six of the seven units were completed as far as the erection and riveting of the scroll cases and pit liners by the time the building cranes were available for the heavy lifts of the butterfly valves. As fast as the valves were in place and riveted, the concrete fill around the scroll cases was poured, and the interior of the generator room completed.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE SHANDAKEN TUNNEL

Discussion*

By Messrs. W. W. Brush and Charles Goodman.

W. W. Brush,† M. Am. Soc. C. E.—The speaker cannot add any information as to the construction work for the Shandaken Tunnel. For the last twenty-two years New York has carried on its water supply work under two separate organizations. The Board of Water Supply has designed and constructed the works for additional water supply, and the Department of Water Supply, Gas, and Electricity has taken over the completed works and maintained and operated them.

The tunnel was indirectly responsible in part for the Ashokan Reservoir being at a very low level on two separate occasions. This statement is not a charge against the tunnel because some of the men in the Department of Water Supply, Gas, and Electricity were directly responsible. The first low level was in 1923, when, to avoid unnecessary pumping, the Department tried to utilize the maximum quantity of water that might be drawn from the Catskill System, with enough left over to take care of the needs of the City until the supply came from the Schoharie through the Shandaken Tunnel. The run-off was low during that year, so that in the fall of 1923 the Ashokan Reservoir was down to about 16 000 000 000 gal. capacity as against 130 000 000 000 gal. when full.

By June, 1924, with the aid of the tunnel, the reservoir was nearly full. In 1926 a low level was again recorded due, once more, to the engineering desire to avoid the expense of unnecessary pumping. This low level resulted in a very sensational newspaper story about leakage from the Ashokan Reservoir. It was stated that the water was going out through the bottom as fast as it came in, and that was the reason why the reservoir was low in the fall of 1926.

* This discussion (of the paper by R. W. Gausmann, M. Am. Soc. C. E., published in May, 1927, *Proceedings*, and presented at the meeting of June 1, 1927), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Engr., Bureau of Water Supply, Dept. of Water Supply, Gas and Electricity, New York, N. Y.

At present it is nearly full, containing more water than at any time since the spring of 1917, and no evidence of a leak has ever been found.

From an engineering viewpoint the story of the leak was absurd, but from the viewpoint of the consumer in the city, who pays very little attention to his or her water supply, except when that supply is interrupted for any cause, it made an unfortunate impression which remains in the minds of many to-day.

It is not an engineering question, but a very human one, whether it was wise to try to save hundreds of thousands of dollars by avoiding unnecessary pumping through lowering the level of the water in the Ashokan Reservoir, thus unintentionally creating in the minds of many a feeling that the system had not been functioning successfully, or whether it would have been wise to spend the money to pump the water and let it go over the dam in the spring of 1927. Apparently, the people of New York would be perfectly satisfied to pay the bill and not save hundreds of thousands of dollars, simply so that they could see a lot of water in the reservoir.

The probabilities are that, with the tunnel and the reservoir completed, and with the consumption in the city increased, it will not be safe to take the reservoir level down to as low a point as it was taken in the fall of 1926. The writer believes that the people in the city would rather pay the cost of pumping some water that would go over the dam rather than save their money, have a low reservoir level, and be asked to conserve water. Although the Department only requested care in use of water, the public thought the Department was trying to develop a water famine for the purpose of accelerating action on an additional water supply system for the city. With water flowing over the Croton Dam yearly, it is impossible to convey to the average citizen of New York the idea that the city needs to go farther afield to get additional water supply. That seems to be a thought that cannot be brought home.

It would manifestly appeal to engineers that if several sources of water supply are available, the obvious thing to do is to operate the system so as to minimize the cost; and, yet, when an engineer tries to serve the several million people in New York City, and to utilize the water-works in the most efficient and economical manner, it seems almost impossible to obtain the public support. When the people are told that they will need additional sources of supply made available ten years hence, many think that this is an exaggerated statement made for the purpose of enabling the engineer to plan and carry out additional works at the cost of the tax-payer.

The people of New York are very proud of their water supply system, and justly so, but they seem to be very suspicious of the need of developing additional sources of supply. The Shandaken Tunnel, prior to the completion of the Schoharie Reservoir in the fall of 1926, furnished an average of 170 000 000 gal. per day for about two and one-half years. It was a great aid in enabling the Department to meet the requirements of the city without installing additional pumping equipment to raise the Croton water to a higher level.

With the completion of the tunnel, the normal available yield of the Catskill System will be about 600 000 000 gal. per day. Just what the maximum dependable yield will be is necessarily uncertain. The stream-flow record in

the Catskills is only for twenty odd years, and that is not long enough to determine what water supply is available; but water supply operations will be on the basis of 600 000 000 per day.

The Catskill Aqueduct will deliver about 630 000 000 gal. per day. It did deliver about 660 000 000 gal. per day in 1925, but with the higher water level now maintained and with the resultant growths on the sides, which have interfered with the flow, the daily capacity is now about 630 000 000 gal. The present delivery capacity is being raised by the use of chlorine to destroy the growths on the inside surface of the aqueduct.

The maximum delivery of the tunnel has been practically 650 000 000 gal. per day (the figure set by the designer), and the normal dependable supply that will be obtained by the combined use of the tunnel and the reservoir will be about 300 000 000 gal. per day.

The Schoharie Reservoir has once been virtually full and then emptied since the fall of 1926, and it is now June, 1927) about half full. It will probably be emptied rather rapidly later in the year. Usually, the Schoharie Reservoir will be at a low level for the purpose of holding the flood flows, because the reservoir is small compared with the capacity of the water-shed. The reservoir has a capacity of only 20 000 000 000 gal.

The work that the Board of Water Supply has done on the Catskill System has been designed and carried out in such a manner that it has met every promise made for it, and all the parts have functioned, from the operating viewpoint, in an exceptionally efficient manner.

Charles Goodman,* M. Am. Soc. C. E.—The trimming of a rock tunnel is made necessary where a minimum dimensioned cross-section is called for; usually it is needed for a timber or concrete structure. This trimming is best done immediately back of the heading before the track, the pump, the air-pipe lines, and the electric light and power lines are laid; otherwise, these must be moved out of the way of the blasting for trimming, thereby incurring additional expense and a loss of time.

Careful setting of line and grade plugs for directing tunnel alignment results in saving rock-trimming work. One plug set to correct line every 50 ft. is better than incorrect plugs every 25 ft.

In driving a heading, the rim line to guide in locating drill holes has to be painted. The question arises as to whether this line shall be painted at the minimum section or 3 to 6 in. wider than the minimum section, in order to avoid subsequent trimming work. If the rock is hard, usually it will blast out in an approximate straight line between the drill holes; and in softer rock, back of the drill holes. The deviation of a straight line between drill holes 30 in. apart from a curved rim of 5 ft. radius is more than 2 in.

In pointing a drill hole, a driller must have some leeway and allowance for the space occupied by the drill; thus, where a tight roof exists, the drill hole must be started possibly 3 in. inside the rim line, and the hole must be pointed carefully to insure that the bottom will not be wide. Experience shows that considerable trimming has to be done when drill holes are placed on

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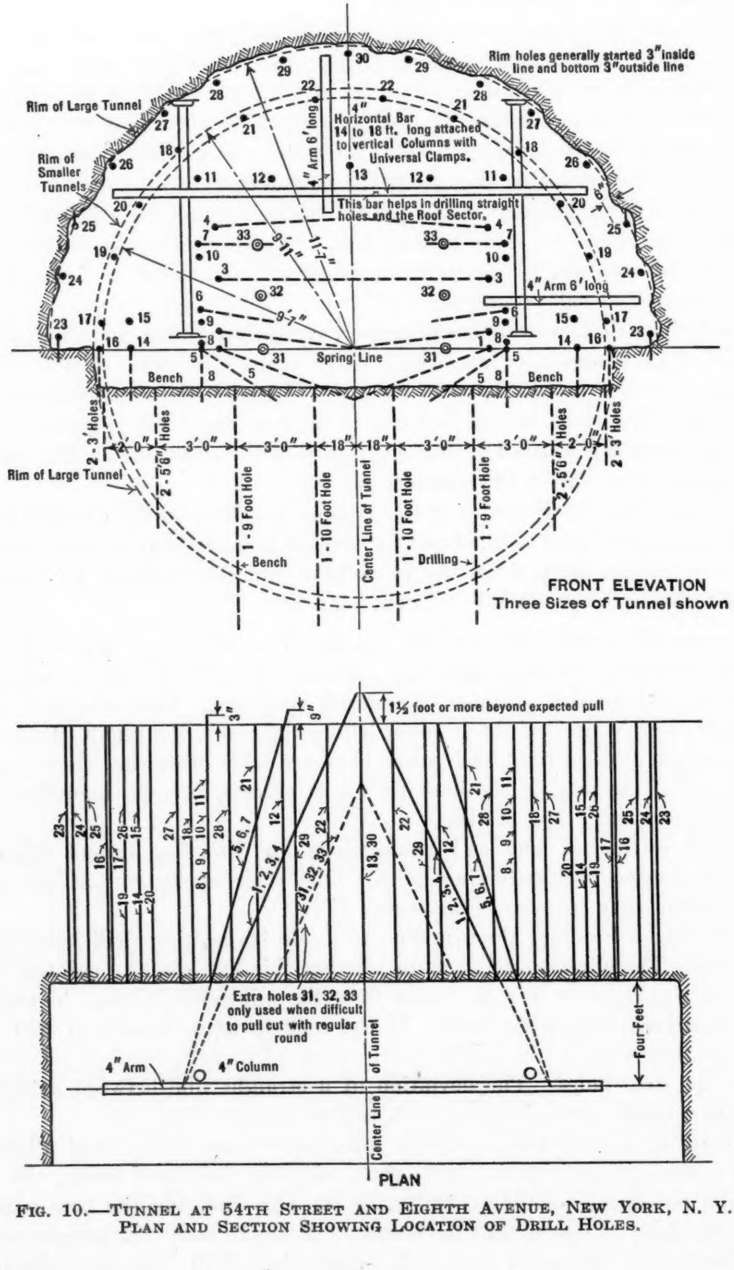

FIG. 10.—TUNNEL AT 54TH STREET AND EIGHTH AVENUE, NEW YORK, N. Y. PLAN AND SECTION SHOWING LOCATION OF DRILL HOLES.

the neat line. If they are placed 6 in. wider than the neat line, the additional cost per square foot of tunnel surface for the extra ½ cu. ft. of rock removed, would be about 5 cents; and for the ½ cu. ft. of excess concrete, about 20 cents; or a total of about 25 cents per sq. ft. The speaker believes that trimming costs would be much more than this and, therefore, recommends that drill holes be located 6 in. back of the neat lines.

The next step is to arrange carefully, so that drilling can be properly done to line. Place the column, whether horizontal or vertical, at a certain regular distance from the rock face to be drilled. The column arms supporting the drills should also be placed at studied locations. Thus, the drill can be easily brought to its correct position from which the hole will be bored in its intended location. Drill holes located straight in or straight down aid in speeding up work and simplify the alignment. Downward bench holes should end at the proper heights, measurements being taken from the column and column arms.

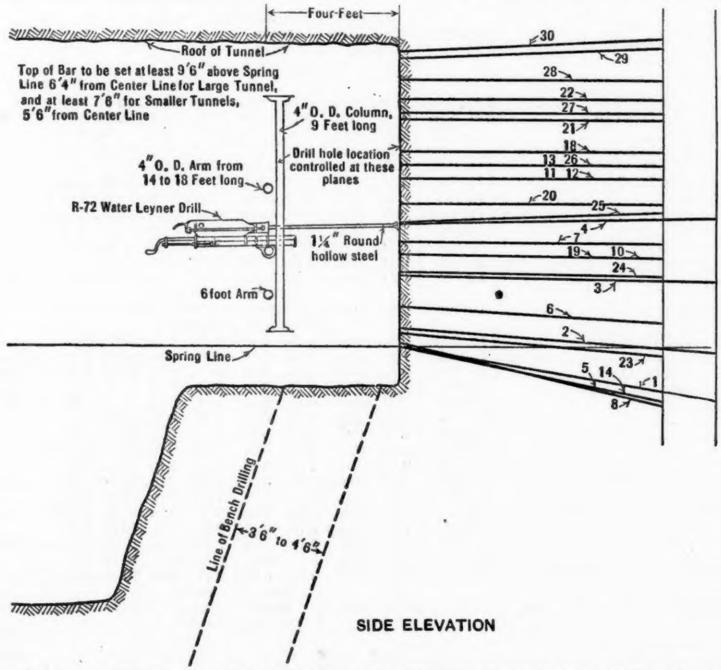

FIG. 11.—TUNNEL AT 54TH STREET AND EIGHTH AVENUE, NEW YORK, N. Y. ELEVATIONS SHOWING DRILL HOLES.

The spacing, depth of holes, and quantity of dynamite charge needed in each hole to break the rock can be best ascertained by results which, in turn, will indicate how the holes should be drilled to eliminate trimming. It is believed that rock miners will not object to using methods which would result in eliminating subsequent trimming, especially if simple mechanical devices for pointing drill holes are used.

Figs. 10 and 11 (with Table 8) show methods used in driving a tunnel 250 ft. long around a curve at 54th Street and Eighth Avenue, New York,

N. Y. The heading rim was painted 6 in. back of a minimum cross-section, and subsequent trimming was not necessary. The average breakage of the heading was 8 in. back of this paint line, or 14 in. back of the minimum line. The down-hole bench drilling was only partly successful, as the bottom broke out in a horseshoe shape, notwithstanding that a number of bottom horizontal holes were used. However, subsequent trimming of the bottom half was not necessary. The average bottom breakage was 17 in. back of the neat line.

TABLE 8.—LOCATION OF DRILL HOLES (SEE FIGS. 10 AND 11).

No. of drill hole.	FOR 10-FOOT SHOT.						Center line to drill, 4 ft. from center.	Length of hole.	Remarks.
	Horizontal Arm Above Springing Line.		Hole Looks:		Face Distance.				
	Drill on top.	Drill below.	Up.	Down.	Above springing line.	Sideways from center.			
1	....	1 ft. 8 in.	....	2 ft. 0 in.	0 ft. 0 in.	5 ft. 2 in.	7 ft. 0 in.	12 ft. 9 in.	....
2	....	1 ft. 11 in.	....	0 ft. 8 in.	0 ft. 8 in.	5 ft. 2 in.	7 ft. 0 in.	12 ft. 9 in.	....
3	1 ft. 11 in	....	....	Level	2 ft. 8 in.	5 ft. 2 in.	7 ft. 0 in.	12 ft. 9 in.	....
4	3 ft. 7 in.	....	0 ft. 4 in.	....	4 ft. 8 in.	5 ft. 2 in.	7 ft. 0 in.	12 ft. 9 in.	....
5	....	1 ft. 11 in.	....	2 ft. 0 in.	0 ft. 0 in.	5 ft. 10 in.	7 ft. 0 in.	10 ft. 6 in.	....
6	....	2 ft. 7 in.	....	0 ft. 6 in.	1 ft. 6 in.	5 ft. 10 in.	7 ft. 0 in.	10 ft. 6 in.	....
7	....	....	....	Level	4 ft. 0 in.	5 ft. 10 in.	7 ft. 0 in.	10 ft. 6 in.	....
8	....	2 ft. 2 in.	....	2 ft. 4 in.	0 ft. 2 in.	5 ft. 10 in.	5 ft. 10 in.	10 ft. 0 in.	....
9	....	....	....	Level	1 ft. 0 in.	5 ft. 10 in.	5 ft. 10 in.	10 ft. 0 in.	....
10	....	....	....	Level	3 ft. 6 in.	5 ft. 10 in.	5 ft. 10 in.	10 ft. 0 in-	....
11	....	....	....	Level	6 ft. 6 in.	5 ft. 10 in.	5 ft. 10 in.	10 ft. 0 in.	....
12	....	....	....	Level	6 ft. 6 in.	3 ft. 0 in.	3 ft. 0 in.	10 ft. 0 in.	....
13	....	....	....	Level	7 ft. 0 in.	Center line tunnel	Center line tunnel	10 ft. 0 in.	....
14	....	1 ft. 11 in.	....	2 ft. 0 in.	0 ft. 0 in	8 ft. 6 in.	8 ft. 6 in.	10 ft. 0 in.	....
15	....	....	....	Level	1 ft. 1 in.	8 ft. 6 in.	8 ft. 6 in.	10 ft. 0 in.	....
16	....	1 ft. 11 in.	....	2 ft. 0 in.	0 ft. 0 in	9 ft. 8 in.	9 ft. 8 in.	10 ft. 0 in.	Leave out
17	....	....	....	Level	1 ft. 0 in.	9 ft. 7 in.	9 ft. 7 in.	10 ft. 0 in.	for 11 ft.
18	....	....	....	Level	7 ft. 6 in.	6 ft. 6 in.	6 ft. 6 in.	10 ft. 0 in.	7 in. tunnel
19	....	....	....	Level	8 ft. 6 in	9 ft. 0 in.	9 ft. 0 in.	10 ft. 0 in.	....
20	....	....	....	Level	5 ft. 6 in.	8 ft. 0 in.	8 ft. 0 in.	10 ft. 0 in.	....
21	....	....	....	Level	8 ft. 9 in.	4 ft. 0 in.	4 ft. 0 in.	10 ft. 0 in.	....
22	....	....	....	Level	9 ft. 6 in.	1 ft. 3 in.	1 ft. 3 in.	10 ft. 0 in.	....
23	....	1 ft. 9 in.	....	0 ft. 9 in.	0 ft. 6 in.	11 ft. 3 in.	11 ft. 3 in.	10 ft. 0 in.	....
24	....	....	....	Level	2 ft. 10 in	11 ft. 0 in.	11 ft. 0 in.	10 ft. 0 in.	....
25	3 ft. 8 in.	....	0 ft. 6 in.	....	4 ft. 9 in.	10 ft. 6 in.	10 ft. 6 in.	10 ft. 0 in.	....
26	....	....	....	Level	7 ft. 0 in.	9 ft. 0 in.	9 ft. 0 in.	10 ft. 0 in.	....
27	....	....	....	Level	9 ft. 0 in.	7 ft. 0 in.	7 ft. 0 in.	10 ft. 0 in.	....
28	....	....	....	Level	10 ft. 2 in.	5 ft. 0 in.	5 ft. 0 in.	10 ft. 0 in.	....
29	9 ft. 11 in.	....	0 ft. 6 in.	....	11 ft. 0 in.	2 ft. 6 in.	2 ft. 6 in.	10 ft. 0 in.	....
30	10 ft. 2 in	....	0 ft. 6 in.	....	11 ft. 4 in.	Center line tunnel	Center line tunnel	10 ft. 0 in.	....
31	....	....	....	....	....	....	....	....	Extra
32	....	....	....	....	....	....	....	....	holes if
33	....	....	....	....	....	....	....	....	necessary

Straight holes, nearly perpendicular to the face, were used for driving the heading of the last half of this tunnel. The cut holes extended at least 1 ft. beyond the expected advance of the round. This cut was placed at the top of the bench. A long 4-in. horizontal bar was attached to two 4-in. vertical columns; 6-ft. arms attached to the columns and to the horizontal bar enabled the drills to be placed close to rim, so that a straight, almost perpendicular, hole could be drilled. This supporting system made a rigid frame on which six or eight drills could be mounted.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

THE EYE-BAR CABLE SUSPENSION BRIDGE AT FLORIANOPOLIS, BRAZIL

Discussion*

By Messrs. F. N. Menefee and T. Kuo.

F. N. Menefee,† M. Am. Soc. C. E. (by letter).‡—The use of heat-treated carbon steel in bridge construction is very interesting. Readers of the *Transactions* of the Society will recall the studies by J. A. L. Waddell, M. Am. Soc. C. E., of alloy steels a few years ago.§ At that time, however, heat-treating was confined almost entirely to smaller units than those usually found in bridges, and was only economical when a large number of small parts were being produced.

The secrecy of the heat-treatment process need not cause any general concern among engineers. There is no longer any real mystery about heat-treatment, and it will soon be found that all the large companies will produce heat-treated bridge members as wanted.

The Brinell test will form a fairly reliable check on the uniformity of the treatment and of the tensile strength. Comparisons may be made against a standard by Brinelling a few full-sized members throughout their length and then testing them to destruction to determine their physical properties.

Considering the fact that stresses in a pin-connected structure, such as a suspension bridge made up of eye-bars, are more accurately determinable than those in a stiff-jointed structure, and that impact, as it is generally understood, on railroad bridges has little or no effect on a bridge the joints of which will yield, as in the case of the suspension type, it would seem that the working stress of 46 500 lb. per sq. in. is well within the limits of safety on steel the elastic limit of which is 80 000 lb. per sq. in.

* Discussion on the paper by D. B. Steinman and William G. Grove, Members, Am. Soc. C. E., continued from October, 1927, *Proceedings*.

† Prof., Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

‡ Received by the Secretary, September 21, 1927.

§ "The Possibilities in Bridge Construction by the Use of High Alloy Steels," *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 1.

Secondary stresses due to uneven settlement of piers do not enter into the stress conditions of the cable portion of the suspension bridge. Axial loading reduces the amount of members which otherwise might have bending moment and shear, and this at once introduces some heavy material in a neutral zone where it does little or no work.

Eye-bars, instead of cable, in the upper chord, will serve two purposes: (*a*) by adding stiffness in vertical and horizontal planes, and (*b*) by decreasing deflection. For the same total area of steel, a cable will elongate under load more than a solid piece of steel.

T. Kuo,* Esq. (by letter).†—In considering the question of rigidity of suspension bridges, one must recognize at the start the characteristic features of two different types of construction with special regard to the make-up of their principal carrying member, the cable. The degree of rigidity attainable in each design depends to a large extent on the question whether the cable is free or braced.

Theoretically, a flexible free cable may be viewed as a structural polygon with an infinite number of hinges connecting bars of infinitesimal lengths. Such a polygon will never constitute a rigid system. While an inverted three-hinged arch would be just sufficient to be stable, the free cable alone, as a structural system, is extremely defective.

Such a flexible cable not only possesses the characteristic property of altering its position of equilibrium with new distribution of loading, but this change of shape, causing excessive deflections in an actual bridge system, has nothing whatever to do with the question of stress and strain. Until the final position of equilibrium is reached, the cable will never be stressed.

Therefore, a free cable suspension bridge is by nature a flexible structure. Even with the stiffening truss, which is designed only to prevent undue deflection, it still retains these basic characteristics to a greater or less extent, depending on the rigidity of the stiffening truss used and the relative magnitude of the live load over the dead load.

When the cable is combined into a rigid system of triangles (braced-cable construction), the result is a stable trussed structure, radically different from a purely free cable suspension bridge made adequate for practical requirements by a secondary stiffening truss. Although this braced cable construction retains much the general nature of a suspension bridge, it lacks the flexible character of an unstable system. All deflections are directly due to the internal deformation of its component members and not to a mere unrestrained movement of a loose system to satisfy a new set of loadings.

One, therefore, must look on the greater flexibility of a free cable suspension bridge somewhat as an inherent characteristic of a certain type of construction and tolerate it as a necessary disadvantage, because of the superior qualities in its other features.

By adopting an unusually heavy and deep stiffening truss, it is possible to design even a free cable suspension bridge to any desired degree of rigidity.

* Structural Draftsman, with Ralph Modjeski, Cons. Engr., New York, N. Y.

† Received by the Secretary, September 26, 1927.

Nevertheless, this departure from the usual proportions set by modern best practice results in a very uneconomical and clumsy design

Moreover, there are other ways of limiting and controlling the deflection of suspension bridges. In the Florianopolis Bridge, the designer has accomplished this in a very scientific and economical manner. The structure belongs to the partially braced type.

On the other hand, deflection is certainly not desirable if it is excessive. Besides the good reasons relating to practical requirements, as set forth in the paper, there is another of more than technical interest for controlling and limiting the deflections in suspension bridges. As the stiffening truss deflects, the floor system deflects with it. This forced deformation on stringers rigidly connected to the girder (except expansion ends), produces secondary stresses that are not taken care of in the stringer design.

Knowing the deflection of the truss, and assuming the curve to be parabolic, the change of end slopes of the stringer and the movement of one end relative to the other, can be readily computed. Then, using the general slope-deflection formula involving these factors, the actual stresses are not difficult to ascertain. The writer has found them to be so extremely high, that they cannot be rightly regarded as being of secondary importance.

While the writer does not doubt the wisdom of using rocker towers in this design for that particular location, he is of the opinion that the use of such towers, like that of pin-connected trusses in ordinary bridge construction, should in general be discouraged.

In time of earthquake, the fixed ended towers are in a much better position to resist the forces produced, and it is quite possible that objectionable displacement between the rocker and the foundation is likely to occur. A rocker tower, due to its construction, is not in integral connection with its supporting pier. When destructive horizontal ground movements are taking place, the inertia of the tower acts against them. Unless the movements are extremely slow, which is hardly the case with an earthquake, the tower cannot be expected to move back and forth along with the pier.

Severe earthquakes are usually infrequent at any one place, and, therefore, elaborate and expensive precautions against them are not to be recommended in all locations. However, in view of the existence of a potential earthquake, even in regions that are generally considered dependably quiet, it seems to be good engineering policy to utilize the superior advantage of fixed ended towers against possible hazard of this kind. Moreover, they involve only a little extra cost.

The new departure of the stiffening truss from parallel chords and the union of the cable with the middle portion of the top chord into a single member are two of the most admirable features of the design. Striving for a more scientific proportion in the height of the stiffening truss and for a more economical structural combination in the make-up of the whole bridge, the designers of the Florianopolis Bridge have established a distinct development in the art of suspension bridge construction. The remarkable results achieved in regard to rigidity and economy of material from this seemingly

theoretical complication has been already comprehensively treated in the paper.

That the height of the truss should vary with the magnitude of the maximum bending moment seems to be one of the broad and fundamental principles of good design. Compliance with it always results in a better design. It is in perfect accord with structural requirement: A longer arm for a bigger moment.

It also converts the outline of the bridge into a more expressive form, which is virtually a modified maximum moment diagram. While it may not be possible for one to write into the structure the story of its conception so that "all who run may read", nevertheless it can be at least plainly felt in the minds of designers that structural fitness could not be more strikingly expressed in other ways.

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PAPERS AND DISCUSSIONS

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THE RELATION OF HIGHWAY TRANSPORTATION TO THE RAILWAY

Discussion*

By M. H. Gerry, Jr., M. Am. Soc. C. E.

M. H. Gerry, Jr.,† M. Am. Soc. C. E. (by letter).‡—American railway transportation is undoubtedly the best and cheapest in the world, nevertheless, it has reached a point where it is too slow, too inflexible, and too costly to meet the requirements of expanding business in this country. The motorized highways have very greatly relieved the situation, and to an increasing extent will continue to do so; but a satisfactory general solution is not likely of attainment save through a re-organization of the railways, and their operation in combination with other facilities now available for public transportation. Perhaps the most important matter confronting industry to-day is that of providing quicker and cheaper means of moving freight and passengers over the great distances separating sections of this country.

The United States presents the largest and the most diversified market for goods and products, and it has also an abundant supply of raw materials, power, and virgin soil. Foreign competition at home can develop only as permitted by insufficient means of domestic distribution.

There is no known limit to the amount of transportation that may ultimately be utilized if made sufficiently rapid and proportionately cheap. The tremendous development of the highways has demonstrated this fact, and furthermore it has shown that speed and convenience both draw and create traffic and that the public appreciates the saving in time and will pay accordingly.

It must be quite apparent to any one who has given the matter serious thought, that there must be continual improvement, betterment, and much new construction in connection with the railroads, or National transportation will

* Discussion on the paper by Ralph Budd, M. Am. Soc. C. E., continued from October, 1927, *Proceedings*.

† Cons. Engr., San Francisco, Calif.

‡ Received by the Secretary, September 8, 1927.

be seriously impaired within the next decade. For a considerable period there has been no general expansion of the railways, at a time when highway development has been the greatest in history. Nevertheless, the rails are still the back-bone of public transportation and are likely to remain so, as no other facilities on land, when properly operated, are their equal for moving dense passenger or freight traffic over material distances.

In recent times the railways have represented about the only great business in the United States that has not shown material expansion. This can be illustrated by Mr. Budd's figures on equipment in use, invested capital, and annual cost of transportation during the past five years. However, there has been substantial progress in other directions. The railroads have handled more freight than ever before and have reduced the time in transit; operating conditions have been bettered and efficiency increased; and their financial position has been improved and their securities are now relatively attractive as investments. On the other hand they have added but little to their facilities or their capital.

Since 1922 the electric and gas public utilities of the United States have placed about \$8 000 000 000 of new securities, while the industrials gained about \$10 000 000 000. For the same period the railways issued only about \$3 000 000 000. There is a reason for this situation, and it can hardly be found in a lack of public demand for the kind of transportation which the railways can best provide; nor can it be found in the impossibility of financing any well-considered development. It may have arisen in part from conservatism in railroad management, or it may be due to Government restrictions; but it is probably the result, in no small degree, of reliance on the fallacy that the total amount of traffic was a fixed entity, controlled only by population and industrial conditions, and not susceptible of material increase by betterment of its own characteristics of speed, convenience, and cost. It has now been proven, however, by experience with motorized highways that traffic, like the demand for electrical power or for merchandise, responds in volume to improved service at lower cost.

There are large sections in the interior of the United States that would profit immensely by quicker and less expensive transportation, and if this can be attained in any way, a great increase in the volume of traffic may be expected, with profit alike to the region served and to the carrier companies. It is probable, therefore, that very soon "public convenience and necessity" will demand much more expeditious and efficient mass-distance transportation, coupled with some form of collection and delivery system operating under one contract. To accomplish these results, new facilities must be used and new methods employed. Thus, the problems involved, to a material degree are those of engineering and a discussion by the profession is, therefore, opportune.

Time in Transportation.—Of all the elements of transportation, except costs, time is the most important. It bears directly on the value of the service and it enters materially into the cost. Most things of importance in the business world run with time: Interest, depreciation, taxes, labor costs, life

itself, are measured in its terms. Every advance in transportation has been based primarily on time. The quickest movement at equal cost will always win, be it for a letter, a passenger, or 1 000 000 tons of coal. The public will pay more for fast service because it is of greater value.

The motorized highways have provided a very much quicker movement of passengers and freight under a great variety of conditions, and, therefore, they have not only taken over from the railways their slow-moving local traffic, but they have created a fast volume of business not heretofore existing. Time enters, not only as speed in transit, but as the full interval required, from readiness to move to arrival at final destination. Therefore, frequent service, regular schedules, convenient terminals, and expeditious gathering and delivery of merchandise, all accelerate transportation and increase its value.

In the movement of passengers, comfort and safety are important factors, but they control the volume of business only when co-ordinated with time. The railroads now provide a high degree of safety and reasonable comfort on most of their trains. It is true, of course, that some "locals" hardly compare in comfort with the best motor coaches, but it is doubtful whether much traffic has been lost on that score alone. The automobile and the motor-bus have diverted traffic from the railroads because they take the passenger over more territory, where he wants to go, with less expenditure of his time, when he is ready to move; and for him this is cheaper transportation. Mr. Budd points out that the buses gain traffic by making frequent stops along the highways, but after all this is only another way of providing a quicker movement of the passenger from the point of origin to destination. Moreover, in California, there are fast bus lines that operate only from depots, make fewer stops than the trains, and better time in transit. With frequent schedules, this service has created for itself a very large volume of business without materially affecting railway traffic in the same territory.

The question is frequently asked, to what extent can the rails compete with the highways for passenger business? The answer is that they can compete wherever the volume of traffic is sufficient to justify a faster over-all service than that possible on the highways, at substantially the same fare. For every distance, there is a certain volume that meets this condition, modified, of course, by local situations.

The motor truck has succeeded also largely because it saves time. Shippers realize that there is value in time and will pay for it. The truck is faster than the rails for short hauls, and in many cases for hauls of considerable distance, due to saving in time otherwise required in transferring to and from terminals and in waiting for train movements. For example, a certain freight shipment in California required 2 days, including the terminal transfers and loadings. The same movement is made by truck in 5 hours, with one loading and one unloading. The truck haul costs more, but it is worth more to the shippers. For short hauls the truck is not only a time-saver, but it is less expensive to operate. However, as in the case of passengers, if the volume of freight be increased sufficiently, a point is finally reached for every distance, except the very shortest, where the rails can move

the traffic in less time and at smaller cost than is possible with trucks on the highways.

In the last analysis, time is the controlling factor in transportation. To industry, speedy delivery of materials and products means a reduction of waste in large measure, due to the lesser amount of equipment and capital required in the business. To the carrier companies, faster movements mean greater volume of traffic handled with substantially the same facilities and labor.

It is said that, by good management since the World War, the railways have accelerated the movement of freight on an average by 25 per cent. This is an excellent showing, but it is not enough to meet the future requirements of American business. However, with equipment and facilities which can now be made available, it is believed feasible to further reduce the time required for completed movements of freight and passengers by at least 50% for average conditions in the United States. This improvement, nevertheless, is not possible without a change of motive power and a re-arrangement of the main railway lines, co-ordinated with motorized facilities on the highways.

Arterial Railways.—Highway development is producing great arterial thoroughfares extending from the Atlantic to the Pacific and from Canada to Mexico and the Gulf. These main traffic lanes are contributing largely to the extent and popularity of highway transportation.

The railways, on the other hand, have provided no such arterial ways, although in their case the need is more urgent. This situation has been caused, in part, by diversity of ownership, by competition for business, or by Governmental restrictions, but, in a larger sense, it is the result of lack of vision in connection with the requirements of continental transportation.

All the principal railway companies are operating main-line through services over their own rails and in various combinations with others, but these arrangements do not qualify as arterial trafficways in the sense here intended. To be truly "arterial" they should drain the territory served and fully utilize other rails, buses, trucks, and all available means, as gathering, forwarding, and distributing agencies. The routes followed of course should be the most direct, regardless of present ownership, and, in the end, the rail companies will find a way, by merger or otherwise, to accomplish this result with full protection to all vested interests.

The arterial railways must be equipped, of necessity, for rapid and expeditious movement of passengers and freight over long distances, leaving to the other rails and the highways the task of collecting and distributing the traffic. Such a service would meet with public favor, and it should reduce the time as well as the cost of long-distance transportation.

Electric Traction.—This subject is introduced here because of its material bearing on the future relation of highway to rail transportation. As previously stressed in this discussion, it is believed that speed is a far greater factor in controlling the volume of traffic than the carriers or the public have ever fully realized. For a long period the railways were supreme in this direction, but now, under many conditions, they provide slower transportation than that furnished by the highways, or even in some cases by the waterways.

It is well known that there are many existing limitations to speed on rails, but their removal does not involve difficulties, either physical or financial, as great as those which have been overcome in the development of motorized highways, in the past twenty-five years. Some will say that the track and roadbed cannot be further improved economically, and that this will limit speed. However, only a few engineers will agree with this view. Others will cite many real or supposed obstacles, but most men will agree that improved motive power and train control are essential if any great increase in speed is attained.

The application of electric traction to trunk railroads has been discussed from many angles for the past thirty years, but mainly from the one of cost of operation under existing railway conditions. The general conclusion among railway men has been that a change in motive power was not justified, and, viewed solely from the standpoint of arrested development, they are probably right. For special reasons, however, electric power has been applied to heavy railroading in a number of cases, under a wide variety of conditions, and the results obtained are ample to establish its possibilities in connection with any great increase of speed in railway operation. The modern steam locomotive is a wonderful power plant, considering its limitations of space and weight, but it is fundamentally unsuited for supplying mechanical energy at the rate necessary for moving trains at materially higher speeds.

It is recognized that there must soon be further material improvements in long-distance transportation. The basic requirements are more speed with less cost, and electric traction offers greater promise of aiding in these directions than any other known agency. There are sound engineering reasons for this statement, based on the highest technical authorities, and fully substantiated by wide experience in both the electrical power and railway fields.

The thought is therefore expressed that the main rail lines should be electrified and highly improved so as to adapt them for very fast and heavy traffic; that the less important rail routes and branch lines should be equipped with rail-motor cars, or Diesel electric locomotives; and, finally, that all gathering, transfer, delivery, and short-haul business should be handled by fleets of trucks and buses operating on the highways.

Summary.—Ample, fast, and cheap public transportation is the most essential of all factors in establishing and maintaining American prosperity.

The movement of freight and passengers is inherently a universal public service and ought to be conducted by corporations or other agencies of National character, functioning under one form of Governmental supervision.

The railroad corporations should extend the scope of their operations to include the highways, the waterways, and the airways.

Arterial railways, electrified and fully equipped for heavy traffic at high speeds, are an essential part of any continental transportation system adequate for future industrial needs.

With improved and unified methods of operation, by utilizing facilities now available, it is commercially feasible to reduce materially the cost of conducting transportation and to expedite the movement of traffic in the United States.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

SOME PHASES OF IRRIGATION FINANCING

Discussion*

By Morrough P. O'Brien, Jun. Am. Soc. C. E.

Morrough P. O'Brien,† Jun. Am. Soc. C. E. (by letter).‡—While Mr. Henny presents the immediate problems of irrigation finance in an admirable manner, it seems that some of the more fundamental aspects of the situation are well worth consideration.

Although the development of the western part of the United States will undoubtedly "strengthen our position in the Pacific Ocean", it will hasten the time when the population will be pressing against the limits of the available food supply. Mr. Raymond Pearl has predicted§ that within a hundred years, the United States will have reached a point in the expenditure of its natural resources and food supply comparable to the present conditions in Europe. The time will come when it will be necessary to develop the sections of the arid States that are suitable for irrigation, but any premature development means economic waste, because the present available wealth would be used in an unprofitable venture. By a "premature development" is meant any improvement completed before the price of the products yielded by that improvement is sufficient to pay all the cost plus the compounded interest.

Contrary to the usual belief, an increase in the food supply per capita with a consequent reduction in prices is not a permanent benefit because, as Malthus has shown quite conclusively, the population immediately responds to this stimulus and in a few years the average condition is about the same as before the change. Previous to the introduction of mechanical industries into Germany, the population was very close to the maximum that the country could support as long as the inhabitants remained predominantly agrarian;

* Discussion on the paper by D. C. Henny, M. Am. Soc. C. E., continued from October, 1927, *Proceedings*.

† Care, Am. Express, Berlin, Germany.

‡ Received by the Secretary, September 8, 1927.

§ "The Biology of Population Growth."

but with the advent of machinery and the development of foreign markets for their goods, the population entered upon another cycle and is now approaching a second peak. Such fundamental changes will undoubtedly occur in the future, but it seems to be rather bad policy to depend on them to maintain a high standard of living. If the great areas of the West are developed only when they are necessary (and the evidence of such necessity should be their ability to pay for themselves) the period of general prosperity will be prolonged. Predictions as to the maximum population that a country can sustain are somewhat futile, but it can be stated, with a considerable degree of certainty, that at some future date the population will have reached a point where a considerable portion of it is barely able to sustain life. Such a condition has always led to wars of colonial conquest—for a "place in the sun"—and although the American national conscience has been against the acquisition and permanent retention of foreign colonies, a change is indicated by the attitude of the United States in dealing with the Philippines and Nicaragua. This country will probably, at some not very distant date, enter the competition for the sparsely inhabited regions of Asia, Africa, and South America.

That the irrigation of a few hundred thousand acres of arid lands is immediately going to involve the country in wars and misery is, of course, ridiculous and it is not the purpose of this discussion to advance any such proposition. However, before deciding to construct projects of such magnitude as those proposed for the Columbia and Colorado Rivers, the question should first be decided as to whether they are really desirable at present or might better be held as a latent supply of wealth for the future.

When irrigation projects, of such magnitude that a single State or group of States cannot finance them, are found to be economically sound, Federal aid should be given, but only under conditions that safeguard the interests of the remaining States. It seems that the only equitable basis for such aid is that the States benefited, guarantee the full cost of the project plus the interest until the debt is paid. Although the principle of interest has been attacked from the ethical viewpoint by some writers, the fact remains that the Government pays interest on nearly all money expended, and this interest represents an important item of the total cost. A sum of money compounded at 4% will be doubled in about 17 years. If an irrigation project does not begin to pay off the original cost in that length of time, the cost to the Government is just double the original outlay. There seems to be no more reason for neglecting the interest charge than the original cost, although most writers on the subject regard the two very differently.

When it is remembered that most of the Federal irrigation projects have not paid their original cost and, in many cases, not even a substantial part of it, the demand for more projects hardly indicates their success, but shows a desire for more gifts from a benevolent and paternal Government. Naturally, the owner of arid land is anxious to have irrigation if all the increase in value will accrue to himself and the entire expense will be borne by the Federal Government. From a rather close observation of one large

project, it would seem that for the most part the settlers were guilty of bad faith. When the project was completed, land values had increased to a point where an additional charge per acre for the cost of the project would have made a profit impossible and yet the land changed hands at this price. If this condition is not explained by sheer ignorance on the part of the buyers, it must have been due to a generally accepted belief that enough political pressure could be brought to bear to prevent the collection of the assessments.

In general, governmental expenditures for internal improvements should be put on a more equitable basis than exists at present. It is sometimes argued that the prosperity of one section of the country reflects upon the prosperity of the whole. To say the least, this contention is difficult to prove, and it is certainly not true of competing regions such as California and Florida. It is extremely unfortunate that the "Sugar Bowl" region of Louisiana has been inundated, but it will be a little unjust if through sympathy a part of the cost of rehabilitation is placed on the sugar-beet growers of Ohio and Michigan, and this will be the result if the Government erects flood-protection works on the Mississippi without a guaranty that the States benefited will ultimately assume the indebtedness. Governmental aid to one of several competing districts without the definite obligation of repayment is doubly unjust, because it gives the one region an immediate advantage and, in addition, the cost of the improvement must be assumed in part by the competing regions.

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PAPERS AND DISCUSSIONS

TIDES AND THEIR ENGINEERING ASPECTS
Discussion*

By Messrs. Edward Godfrey, William Barclay Parsons, T. Kennard Thomson, H. F. Dunham, and Ralph Bennett.

Edward Godfrey,† M. Am. Soc. C. E. (by letter).‡—The writer agrees with Commander Rude that the subject of the tides is one that ought to be discussed by engineers, particularly that phase that deals with their cause. Some scientists deal chiefly with speculation. They are not concerned with producing results; nor are they concerned with rigid proofs that certain results will be produced by certain causes. A motor, or engine, or turbine must deliver the power for which it is designed. Hence, the engineer must know definitely, what certain elements, when brought together under certain conditions, will do.

The foregoing remarks are a preliminary to a challenge of the universally accepted theory of tides, which omits mechanical principles that govern the movement of bodies, and glides over adverse facts that nullify the theory completely. Forces of gigantic magnitude are ignored, and secondary forces are given the leading, in fact, exclusive, consideration.

The theory of tides, as expounded by George Howard Darwin,§ at greater length perhaps than by any other writer, is that they are the result of the gravitational pull of the moon on the water of the oceans. This is explained by the author, except that it is the centrifugal force of the earth's rotation, combined with the moon's lessened pull, that causes the tidal rise on the opposite side of the earth from that where the moon is on the meridian.

Writers on tides, including Commander Rude, show the bulges representing the high tide, directly under the moon and on the opposite side of the earth. They also state, as Commander Rude has said,‖ that in Nature it is different,

* Discussion on the paper by G. T. Rude, M. Am. Soc. C. E., continued from October, 1927, *Proceedings*.

† Structural Engr., Pittsburgh, Pa.

‡ Received by the Secretary, August 31, 1927.

§ "The Tides and Kindred Phenomena in the Solar System," by George Howard Darwin.

‖ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1079.

diametrically different, so to speak. The tides are perpetually low on the sides of the earth where the moon is pulling for high tide, and *vice versa*. The only explanation of this astounding nullification of the theory is that the tide lags or is retarded, presumably by the wave trying to catch up with the force which produces it. It is here that a study of engineering mechanics would be a good thing for the scientist. It would be interesting to have him answer the following questions:

(1) Is the force of gravitation itself so sluggish that it takes 6 or 8 hours for it to be felt across the distance from the moon to the earth?

(2) Does not the tidal wave traverse the circumference of the earth in about 25 hours?

(3) Why does this wave not synchronize itself with the force that produces it?

(4) If gravitation is instantaneous, or nearly so, when and how is a tidal wave initiated and what keeps it going, since the force is admittedly always in an entirely different direction from that in which, mechanically, it would have to be to raise the water?

(5) Is there any action in a tidal wave analogous to flywheel action?

(6) Would a wave continue to travel around the earth from east to west, the crest and trough being 6 000 miles apart, if the moon's force were suddenly to cease?

(7) Would not that wave divide and seek a level by traveling both east and west?

A large pendulum can be made to move in wide amplitude by impulses applied by pulling at intervals on a small rubber band, if the timing of the impulses agrees with the swinging of the pendulum, in the same direction as the impulse. However, the pendulum cannot store up impulses applied contrary to its motion to be used later when it is swinging that way. Therefore, it is inconceivable that the ocean could store up, for 6 or 8 hours, a wave-raising force received from the moon and then respond to that force.

If the earth were absolutely rigid, there would be tides due to the moon's gravitational pull, but high tide and low tide would be just about exactly reversed.

The earth is not rigid and does not behave as a rigid body, in spite of the opinion of scientists. If it were a solid steel ball, of the hardest steel known, the gigantic force of the moon would of necessity elongate its diameter in the direction of a line to the moon. The moon's gravitational pull on the near hemisphere of the earth is greater than that on the far hemisphere by about 600 000 000 000 000 tons, a force that would snap a piano wire 30 miles in diameter. Is this a trifling force that can be ignored because some scientist states that in his opinion the earth acts as a rigid body in the face of the moon's tidal pull?

This force distributed uniformly over the entire cross-section of the earth considered as a solid steel bar, 4 000 miles long, would stretch it out 4 or 5 ft. Other metals and rocks do not approach this rigidity. This is not conjecture. There has never been a piece of material found in the earth that is

not subject to this law and would not be elongated by this amount or a greater amount under a force of this intensity.

Furthermore, having been elongated by this force, applied in a constantly changing direction, the elongation would not cease when force and elasticity were equal. Inertia would carry the earth several feet more. A high wave, therefore, in the body of the earth itself is carried around by the moon and what is the result? Of course, a wave of water follows in the trough of the earth wave. This is the ocean tide as it is known, not a wave of water raised by the moon, but a wave of water following and remaining in the trough of the solid earth created by the gigantic differential gravitational pull of the moon.

The tidal pull, rightly understood, accounts for countless phenomena, not otherwise explained in any rational manner. Some of these phenomena are the following:

(*a*).—*Earthquakes.*—The continual kneading of the earth by its satellite accounts for the thousands of earth tremors observed yearly.

(*b*).—*Volcanoes.*—Extra movement with attendant friction below the surface of the earth creates heated spots, where peculiar rock formations exist that are conducive to movement.

(*c*).—*Hot Springs.*—The same process that produces the volcano, working on a small scale produces the hot spring.

(*d*).—*Internal Heat of the Earth.*—The heat of friction due to the continual kneading of the body of the earth by the pull of the moon and the earth's rotation accounts for the internal and absolutely constant heat of the earth. It seems strange to state that the moon and not the sun is responsible for the unfailing supply of heat in the earth, making the planet livable, particularly at night, when radiation of atmospheric heat would leave the air excessively cold.

(*e*).—*Trade Winds.*—The tidal pull of the moon on the earth's atmosphere, apart from land or ocean tides, accounts for prevailing winds, trade winds, etc.

(*f*).—*Ocean Currents.*—Instead of ocean currents being caused as claimed by trade winds, they are doubtless due to the continual travel of a wave of water forced around the equatorial region and deflected by the continents in the manner exhibited by ocean current paths.

The earth's density is several times that of rock. It is, in fact, nearly as great as steel. The interior, therefore, must be metallic. No doubt, it is largely composed of iron or steel.

This great elastic ball, subject to the enormous gravitational pull of the moon, would exhibit inertia in its tidal wave in a manner not possible in a water wave assumed to be actuated by the moon's tide-raising force. Accelerated impulses, such as those of the combined pull of the sun and moon as they approach conjunction or opposition, cause higher and higher tides, and the highest tide, on account of the inertia, would be a little later than the maximum impulse. This accounts for the fact, as pointed out by Commander

Rude, that the highest tide occurs a day later than the actual time of new or full moon.

The frictional heat of the earth, due to tides, is generated comparatively near the surface, probably largely where the interior elastic metallic ball joins the exterior rock crust. The temperature at the center is probably not very great; not much if any greater than the temperature a few miles from the surface. Hence, the metallic core is elastic and not fluid.

Volcanoes are the result of heated spots where extra motion and friction are manifest; they are not an indication of the earth's general internal condition.

The moon's excessive low temperature is due to the fact that it does not rotate as does the earth; therefore, it can have no tide. Its atmosphere and water vapor have turned into snow.

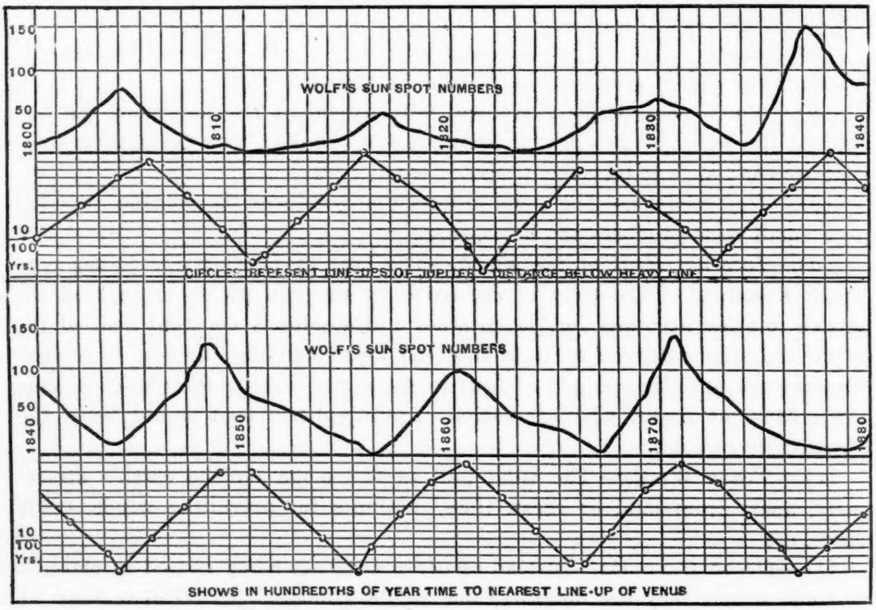

FIG. 39.—GRAPH OF SUN-SPOT INTENSITIES.

The sun is subject to tides of gigantic force. Any one of the four planets, Mercury, Venus, the Earth, and Jupiter, exerts a tidal pull on the sun many times the tidal pull of the moon on the earth. The kneading of its great mass creates heat that is incalculable. It is this that sustains the heat of the sun. A very striking proof of this is exhibited in sun-spot counts for a period of 300 years. Sun spots are volcanoes in the sun caused by excessively heated spots, due, as in the earth, to sub-surface friction.

Astronomers have recognized that there is some general influence, over a long period of years, that intensifies sun spots in a regular period of 11.2 years and some influence that introduces an irregularity, so that the several periods are more or less than 11.2 years. The writer has discovered that there are

exactly such influences at work and that the period of high sun-spot intensities agrees remarkably with those influences.

Every 11.2 years Jupiter and Venus and the Earth are almost exactly in line, thus adding their tidal pulls on the sun and intensifying its heat. Fig. 39 shows the agreement between these alignments and the high sun-spot periods. The influence that makes for irregularity is Mercury, which aligns itself with one or two of these planets in short periods.

Forty-one of Jupiter's half synodic periods is 22.387 years and twenty-eight of those of Venus is 22.381 years. Thus, every 22.38 years these planets and the Earth are within 2 days of being exactly in line. In the intermediate period these planets are twice within 7 days of being in line, as will be seen by comparing their synodic periods.

The argument that will be used against this theory is, "What about the law of the conservation of energy?" That "law" was broken down when radium was discovered. Now, however, to cover the face of the law, scientists assert that all matter has potential energy of almost incalculable amount residing in it, although they do not hesitate to calculate how long a pound of metal would run a horse-power engine, in millions of years. Let them turn engineers for a while and try to lift a fly 1 ft. by a ton of this metal in any period without using some regular old-fashioned coal, wood, or other fuel that is said to have potential energy stored in it, or by using sun radiations.

Energy is inexhaustible and is all derived from the moving, impinging, vibrating ether, traveling at the speed of light and penetrating matter without hindrance. On the earth there are limited means of converting this energy into useful work. Nature's great tidal engines must be depended upon to supply and store up the useful energy needed.

It may seem strange to set up a theory that the heat of the earth is derived chiefly from the moon and that the earth contributes immeasurably more heat to the sun than it receives from the sun; but let any one who is skeptical determine the known forces, from the law of gravitation; one that would snap a piano wire 30 miles in diameter and the other that would snap a similar wire about 150 miles in diameter, each exerted on bodies of enormous size in a different direction every hour. Calculate the horse power needed to accomplish this and the heat that would result, and some of the greatest puzzles of science will be at least partly solved. These puzzles are the undiminished supply of heat from the sun, the constant temperature of the earth, the cause and reason for the periodicity of sun spots, the cause of earthquakes and volcanoes, and, in fact, directly or indirectly, all the mechanical manifestations on the earth or sun, as well as the utter absence of them on the tideless moon.

William Barclay Parsons,* Hon. M. Am. Soc. C. E. (by letter).†—This paper is an admirable exposition of a very complicated natural movement, on some of the aspects of which scientific men are not in agreement. Engineers, however, are not directly concerned with these differences in opinion in the planning and execution of their work. From the standpoint of science they may

* Cons. Engr., New York, N. Y.

† Received by the Secretary, September 6, 1927.

be interested in what is supposed to take place in the great tidal movements and oscillations in the deep open oceans, but in practice all they need know is what will occur under various conditions in harbors, estuaries, or other places along a coast exposed to tidal effect.

The most important factor for the engineer thoroughly to understand is that he cannot generalize with safety in respect to what will take place in tidal action in any one locality. He may know, with nice exactness, the effects and movements at two places, one each side of where he is working, but he must not assume that the result at his site will be a mean of the other two. Local conditions have a tremendous, and frequently surprising, influence. There may be two progressive tidal waves each acting freely and with normal ranges and times, at the other places, but which, meeting and interfering, will give an apparently wholly unrelated result at an intermediate station. Tidal actions are also seriously affected by ocean currents as well as river flow and particularly by the depth and shape of bays and estuaries.

The writer recalls one experience where mean sea level had been previously determined for two places only eight miles distant, one from the other, by measuring high and low-water elevations at both places, taking the mean, and connecting such means with a chain of levels. Being on opposite sides of a peninsula, the places were affected by two tidal waves of different characteristics. Although the levels showed a considerable difference in elevation between the repufed mean sea levels, the elevations had been accepted as accurate. The error was not discovered until the writer had established tide stations, when it became apparent that whereas mean tide at one station was the same as mean sea level, such was not the case at the other. At the latter station there was a large body of comparatively shallow water so that, when the ebb tide was half gone, sufficient frictional resistance began to be developed to retard the flow, and, consequently, the ebbing tide never was able to reach the true low-water level. In this case mean tide and mean sea level were far from being the same.

The engineer should never accept any tidal statements as true unless they are based on accurate observations with self-recording gauges conducted through at least one lunar cycle, probably analyzed and interpreted; and even then he must be on his guard against exceptional tides and prolonged wind effects.

The author refers to the currents in channels joining two bodies of water in which there are tides with different characteristics, and cites as examples the East River, New York, and the Cape Cod Canal. The former, although highly interesting and important, is not a good example from which to draw conclusions, on account of its variable cross-section and sharp turns. The latter on the other hand is almost perfect for study, with its uniform cross-section and only two curves with long radii, well protected from wind influence. Some of the results, as deduced by the writer, were given in his paper entitled "The Cape Cod Canal".* Among other interesting data were (*a*) the curved and not straight line of low-water elevations with an upward central versed sine of about 1 ft., a factor of economic importance; and (*b*) the evi-

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (December, 1918), p. 1.

dent momentum of the moving body of water which causes an actual flow of water up a gradient of measurable slope. This momentum accounts, in part, for the fact that the current in tidal waters does not change direction at the same time as the tide turns.

When the Board of Advisory Engineers, of which the writer was a member, was considering the type of canal for Panama, it refused to discuss or attempt to determine the rate of current that would exist if a canal were built there at sea level without locks, being satisfied with a general statement that the current would not be strong enough to interfere with traffic. So far as the writer is aware no analysis of this question has ever been published.

T. Kennard Thomson,* M. Am. Soc. C. E.—It is fortunate for New York that the State of New Jersey did not have the benefit of Commander Rude's professional services generations ago, when New Jersey claimed Staten Island, because, it was stated, a ship could not sail around it in one day without running aground.

Unfortunately for New Jersey, New York had a captain who knew the tides and channels, and who by starting at the right place, at the right time, was able to get around without landing on a sand-bar.

Fig. 40 shows that the St. Lawrence River, Bay of Fundy, and Long Island Sound are practically parallel, although in opposite directions, and have enormously greater tidal ranges than Chesapeake Bay, Delaware Bay, and the Hudson River, which are also parallel to each other, but approximately at right angles to the first three.

The Bay of Fundy, with no outlet at its upper reach, has the greatest tide, about 47 ft., with an average, as Commander Rude has indicated,† of 5 ft. higher tide on the Nova Scotia side, than on the New Brunswick side.

At Quebec, Que., Canada, the St. Lawrence River has a maximum tide of about 20 ft. and the influence of the tide is slightly felt at Montreal, where the surface of the river is about 24 ft. above mean tide level. The St. Lawrence, above Montreal, receives an enormous fresh-water flow of about 300 000 cu. ft. per sec. from the Great Lakes. This is a comparatively uniform flow.

On the other hand, Long Island Sound has two outlets, the East River and the Harlem River, with a maximum average tide of 7½ ft.; only about 3 ft. more than the average tide at Governors Island.

It would be interesting to know whether a new East River from Flushing to Jamaica Bay and a new Harlem River from Hell Gate to the Hudson, 2.50 miles long instead of 6.75 miles, would reduce the difference between the tide levels at Governors Island and Hell Gate, if each of the new rivers were, say, 50 ft. deep, and of ample width.

Fortunately for the Cities of New York, Philadelphia, Pa., and Baltimore, Md., they are not troubled with such excessive tides, and it would also be interesting to know whether this good fortune is due to the fact that they are at right angles to the direction of Long Island Sound, Bay of Fundy and the

* Cons. Engr., New York, N. Y.

† *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1109.

St. Lawrence; and also whether, if all these rivers, bays, and sounds were running parallel to the Equator, the tidal ranges would be greatly increased.

Chesapeake Bay has a tide of only 1.2 ft. at Baltimore; Delaware Bay has a tide of about 5.2 ft. at Philadelphia; and the Hudson River, a tide of 4.5 ft. at Governors Island, with an ordinary high tide of 2 ft. at Albany, N. Y. The Hudson River, however, is subject to greater fresh-water flood variations than the others. For instance, the speaker has seen a low-water flow of only 600 cu. ft. per sec. in the Mohawk River near its entrance to the Hudson River, and, at the same place, he has seen a flood flow of 100 000 cu. ft. per sec. At The Narrows, where the normal flow is 76 000 cu. ft. per sec., an exceptional flood flow has amounted to between 300 000 and 400 000 cu. ft. per sec. While the flood-flow elevation is not proportionately great at Governors Island, it amounts to 18 ft. at Albany.

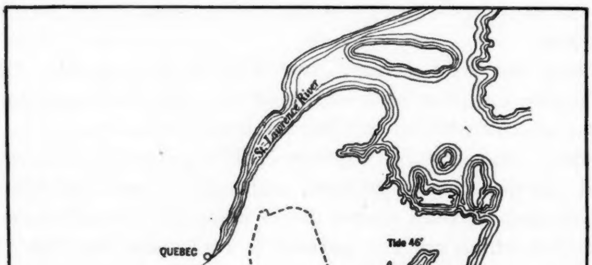

FIG. 40.—GRAPH SHOWING PARALLELISM OF ST. LAWRENCE RIVER, BAY OF FUNDY, AND LONG ISLAND SOUND.

This paper is of great personal interest on account of the plan the speaker has been advocating for more than 16 years for a "Really Greater New York". When he first suggested extending Manhattan down the Bay (where it is hoped to reclaim 9 sq. miles), nearly every one thought that the one insuper-

able objection would be the "tidal prism theory", and it looked as if it might be necessary to construct a model, including New York, New Jersey, Connecticut, and all the fresh and salt-water flows that affect New York Harbor. Such a model, of course, would cost many thousands of dollars, but when a (then) U. S. Chief of Engineers stated that he had no use for the "tidal prism theory" in this respect, the speaker cheerfully discarded the idea of the model.

While in 1911 many laughed at this project for a "Really Greater New York", which included reclaiming a section of the East River, a number of people have recently proposed it as an original idea of their own. One went so far as to say that he had the approval of the War Department, which, of course, was absurd, because he had omitted the first essential feature of the original plan, that of providing an outlet for Long Island Sound to the Hudson River and Jamaica Bay before attempting to close the East River. He did not even pay any attention to Newtown Creek.

In addition to avoiding the danger of turning Long Island Sound into a Bay of Fundy, it is equally important to provide new docks, etc., for the people before closing off 5 miles or more of the East River.

It may be interesting to record the fact that some of the biggest financial men of the country declared that they would not be interested if the East River section of the plan was omitted. This was due (in 1913) to the fact that they realized that New York can never get enough bridges and tunnels to connect Long Island with Manhattan, and also to the relief which this plan will afford the north and south traffic.

Commander Rude has referred to the hydro-electric features of the Bay of Fundy.* Two projects have recently appeared in the daily papers. One plan is for Passamaquoddy Bay between Maine and New Brunswick, of which more than three-quarters would be in Canada, with a maximum tide of 27 ft.; and the other plan is for the upper end of the Bay of Fundy, all in Canada, with a maximum tide of 47 ft.

It is claimed that the latter project would only cost one-half as much per horse power developed as the first plan, but the first promises to develop power at a cost of about four times as high as the speaker's estimates per horse power developed for a "Niagara Falls Junior". There is no immediate prospect, therefore, of the Bay of Fundy developments. However, sooner or later, engineers will have to learn how to utilize these tides economically, storage batteries not being economical; for when all the rivers are fully developed there will still be more coal used than at present, and before that time there is hope of having a new city of 9 sq. miles built in the very heart of New York City, between the Battery and Staten Island, where people will never use a pound of coal, nor any oil or gas.

H. F. Dunham,† M. Am. Soc. C. E.—The author's well organized and excellent paper will be widely read and appreciated. The "Nomenclature" is a fortunate introduction. The definitions‡ offer a slight difficulty to one unaccustomed to these terms, namely, "perigee" and "apogee", termed "posi-

* *Proceedings,* Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1104.

† Civ. and Hydr. Engr., New York, N. Y.

‡ *Proceedings,* Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1072.

tions of the moon in its elliptical orbit around the earth". "Elliptical" as frequently used implies a return to the point or position first occupied by a moving body. In Fig. 41 the lines represent the orbits of the earth and moon on a scale of 2 000 000 miles to 1 in. Is there an elliptical orbit for the moon around the earth in addition to its elliptical orbit around the sun, similar to that of the earth around the sun?

FIG. 41.

Low-tide elevations alongshore where construction work is going forward are complicated with ground-water heights and do not require extreme accuracy. The limit for timber submergence is rather a matter of good judgment. If sub-drains or subways are to follow, the call is for better judgment.

In 1922, the Carnegie Institution of Washington published "Effects of Winds and Barometric Pressure on the Great Lakes". Tidal influences were not recognized; instead, from many observed gauge readings at various points, the 13-hour rise and fall was attributed wholly to wind and barometric pressures.*

In 1903 a tide-gauge was established near the mouth of the Menominee River, on Green Bay, Wisconsin,† where conditions for accurate records were extremely favorable. An unused water intake of metal about 6 ft. in diameter, perforated with ten thousand ⅜-in. holes and placed in 20 ft. of water, was connected by a 12-in. pipe line, 800 ft. long, to a vertical 16-in. pipe well having a float carrying a rod and pencil, which traced a line on a vertical clock cylinder. The pipe line and well were connected by about 20 ft. of 1-in. pipe. Wave effects, so-called, were not recorded as Fig. 42 will show.

Fig. 43 covering a later date is quite exceptional. While wave action may be inferred at first glance the time intervals of many minutes shown by the inequalities of the curves indicate an up-and-down motion of large areas of the water surface. The U. S. Weather Bureau records at the nearest station, Green Bay, Wis. and Escanaba, Mich., show that the wind was southwest and not in excess of 6 miles per hour during the night covered by the record. This shows that the small irregularities on the chart were due to variable barometric pressure. Still another record is given in Fig. 44 (*a*). Below it (Fig. 44 (*b*)), for comparison, are shown the high and low tides for Sandy Hook covering the identical period. The similarity of the two graphs is marked. It is difficult to accept a conclusion that denies or seriously questions the existence of sun and moon influences as distinct from atmospheric effects on the tides of the Great Lakes.

The illustration of a "stationary tide" by the slight movement of a basin containing water,‡ seems especially fortunate if it can be extended to ocean

* "The Manual of Tides," by R. A. Harris, U. S. Coast and Geodetic Survey Report for 1907 was cited as authority.

† Records from this gauge, faithfully kept for many months by the late J. J. Campbell, have been deposited with the U. S. Coast and Geodetic Survey, at Washington, D. C.

‡ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1081.

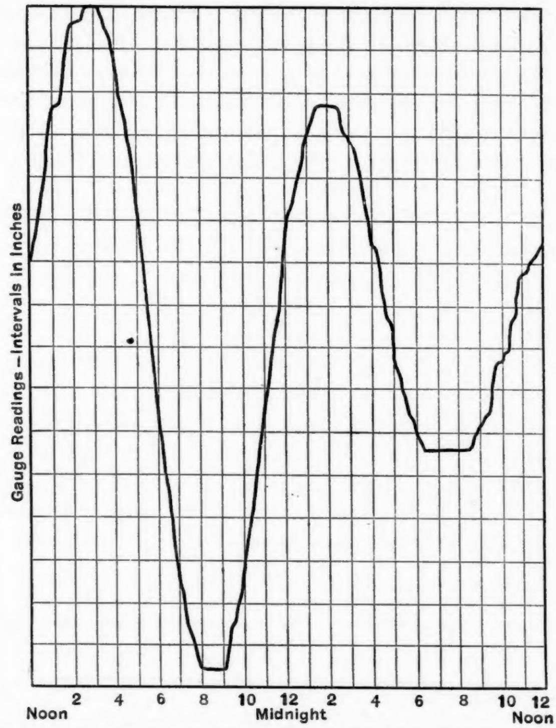

FIG. 42.—VARIATION IN WATER ELEVATION, GREEN BAY, WISCONSIN, DECEMBER 29 AND 30, 1903.

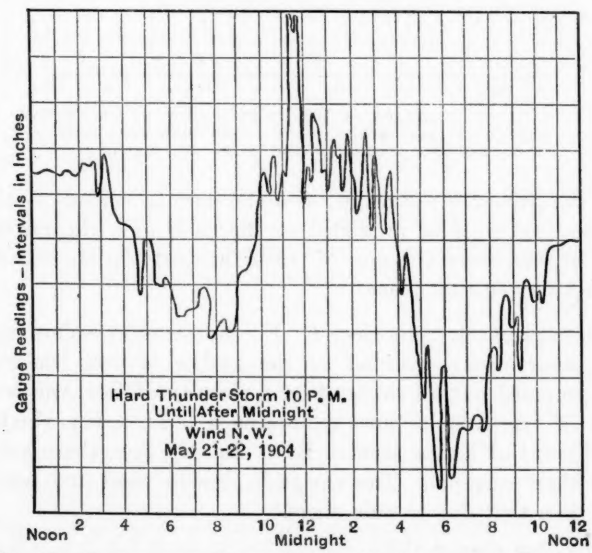

FIG. 43.—VARIATION IN WATER ELEVATION, GREEN BAY, WISCONSIN, MAY 21 AND 22, 1904.

basins and checked with astronomical data. Zenith and tower telescopes disclose a failure to fix position on the earth's surface (the latitude, for instance), closer than a fraction of a second of arc, or less than a horizontal distance of 100 ft. It has long been suspected that the earth quivers a little even at the best sites of observatories, usually on mountain ranges. Various suggested causes have included movements of glacial masses hundreds of square miles

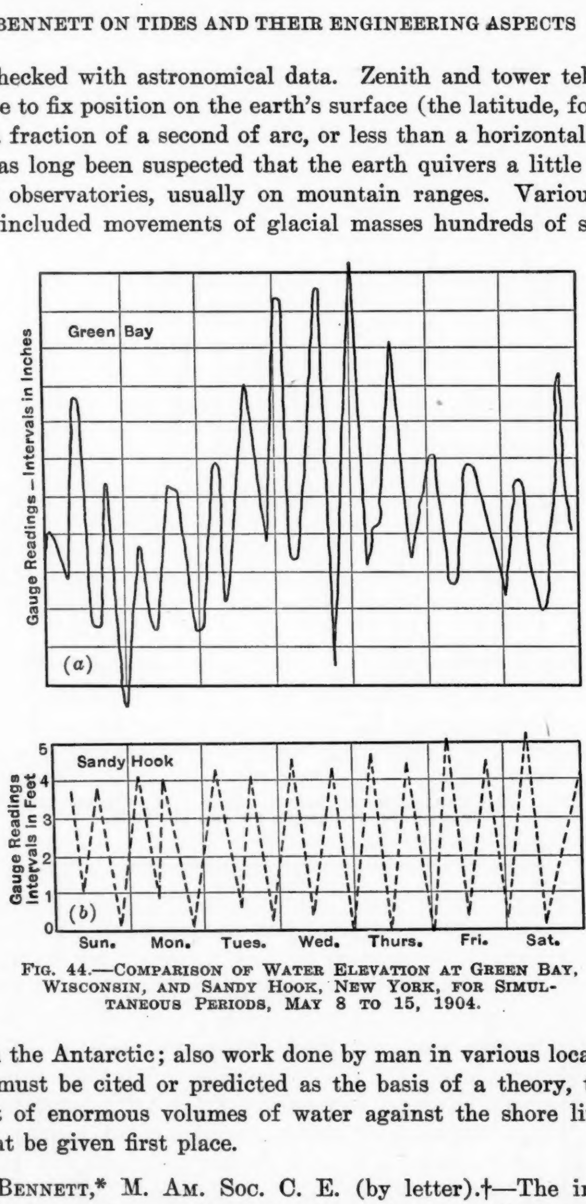

FIG. 44.—COMPARISON OF WATER ELEVATION AT GREEN BAY, WISCONSIN, AND SANDY HOOK, NEW YORK, FOR SIMULTANEOUS PERIODS, MAY 8 TO 15, 1904.

in extent in the Antarctic; also work done by man in various localities; but if something must be cited or predicted as the basis of a theory, the rhythmic tidal thrust of enormous volumes of water against the shore lines of ocean basins might be given first place.

RALPH BENNETT,* M. AM. SOC. C. E. (by letter).†—The importance of the current observations outlined by the author is well illustrated by the present and proposed outfall sewers in the vicinity of Los Angeles, Calif.

The City of Los Angeles now maintains a large sewer which discharges near the mid-point of Santa Monica Bay. In this Bay, there are very feeble and variable tidal currents. The variation due to wind and season is seemingly larger than that due to tide stage.

* Cons. Engr., Los Angeles, Calif.

† Received by the Secretary, September 26, 1927.

Under the condition that existed while the old outfall in the same location was operated, most of the floating matter and much raw sewage drifted on the adjacent beaches soon after discharge. As these beaches are extensively used for bathing, the City was compelled to make much more complete provision for screening and diffusion than had been the case in the old sewer.

In the absence of any extended data as to the total flow of water past the discharge point, the new outfall was designed to produce dilution of the sewage by the use of numerous distributed outlets. The operation of the sewer is under the control of the State Board of Health and any serious increase in objectionable matter on the beaches will result in orders to treat the discharge fully and to release only clear, sterile, and odorless effluent.

The adjacent areas in Los Angeles County are quite largely sewered by the County Sanitation Districts which propose to discharge in a somewhat similar way into San Pedro Channel.

At the point of proposed discharge there is a more definite tidal flow than in Santa Monica Bay, but here, again, reliance will be placed largely on deeply submerged outlets in multiple. This location has been bitterly fought and only because of necessity has the State Board of Health permitted the construction. The sewer may ultimately be equipped for full treatment.

If, in either case, a continuous record of the direction and velocity of the ocean currents at the discharge points could be obtained, a great increase in accuracy of design might be made.

This is very noticeable in the outfall sewer of the Sanitation Districts which has been located without scientific aid. If records of the channel currents were available, it is entirely possible that a point of high velocity and seaward discharge could be found, at which a better and yet less expensive outlet could be located.

A fully continuous record is required, however, in such quiet waters as those of Santa Monica Bay. The device should be capable of producing an automatic record of both direction and velocity for a long time without attention.

A similar degree of performance has been attained in the modern curve-tracing gauges. Long-distance wind direction and velocity recorders are in use. A similar long-distance equipment, actuating a shore-located recorder from an anchored vane, deflected by the velocity of the current and swinging with the direction, can certainly be developed if the market demands it. The need is obvious. Cannot the demand be met?

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

NOTES ON ARCHED GRAVITY DAMS

Discussion*

By Messrs. Lars R. Jorgensen and P. Wilhelm Werner.

Lars R. Jorgensen,† M. Am. Soc. C. E. (by letter).‡—This is not the first time that the author has called attention to the fallacy of arching a gravity dam in plan expecting an increase in stability of the structure therefrom at all seasons. Mr. Jakobsen has shown that a gravity dam should be straight in plan for maximum safety, with a minimum expenditure of material.

If a gravity dam possess a factor of safety of two under the worst conditions of uplift, ice pressure, etc., it has been regarded as a sufficiently stable structure, and probably the future will not change anything in this respect.

Fig. 5 which was prepared for comparing the cross-sections of the highest gravity dams, shows that there is a wide difference in the various profiles. The Kensico Dam, constructed by the Board of Water Supply of New York, N. Y., apparently is the most conservative, and the Exchequer Dam, built by the Merced Irrigation District, California, the least conservative, of all high gravity dams thus far constructed. Between these two extremes is the O'Shaughnessy Dam at Hetch Hetchy, Calif., and the Camarassa Dam, in Spain. These two dams are actually a little more conservative than their relative location on the diagram would indicate, because ice pressure is negligible for both. The two sections nearly coincide and could well be termed standard gravity sections as they represent the middle between the most and the least conservative in high dams. The safety of gravity dams having sections similar to or greater than the two named cannot be questioned, but not all projects can stand the expense of such heavy sections and, consequently, the safety is reduced somewhat to suit the purse of less fortunate projects. It is a fact that in many places where slim gravity sections have been built much safer arch dams could have been substituted for less money.

* Discussion on the paper by B. F. Jakobsen, M. Am. Soc. C. E., continued from October, 1927, *Proceedings*.

† Cons. Engr., Constant Angle Arch Dam Co., San Francisco, Calif.

‡ Received by the Secretary, August 29, 1927.

Some years ago the Engineering Profession was very much divided as to the relative merits of gravity and arch dams; this is still true although to a much less extent. At times, this controversy has resembled a political issue.

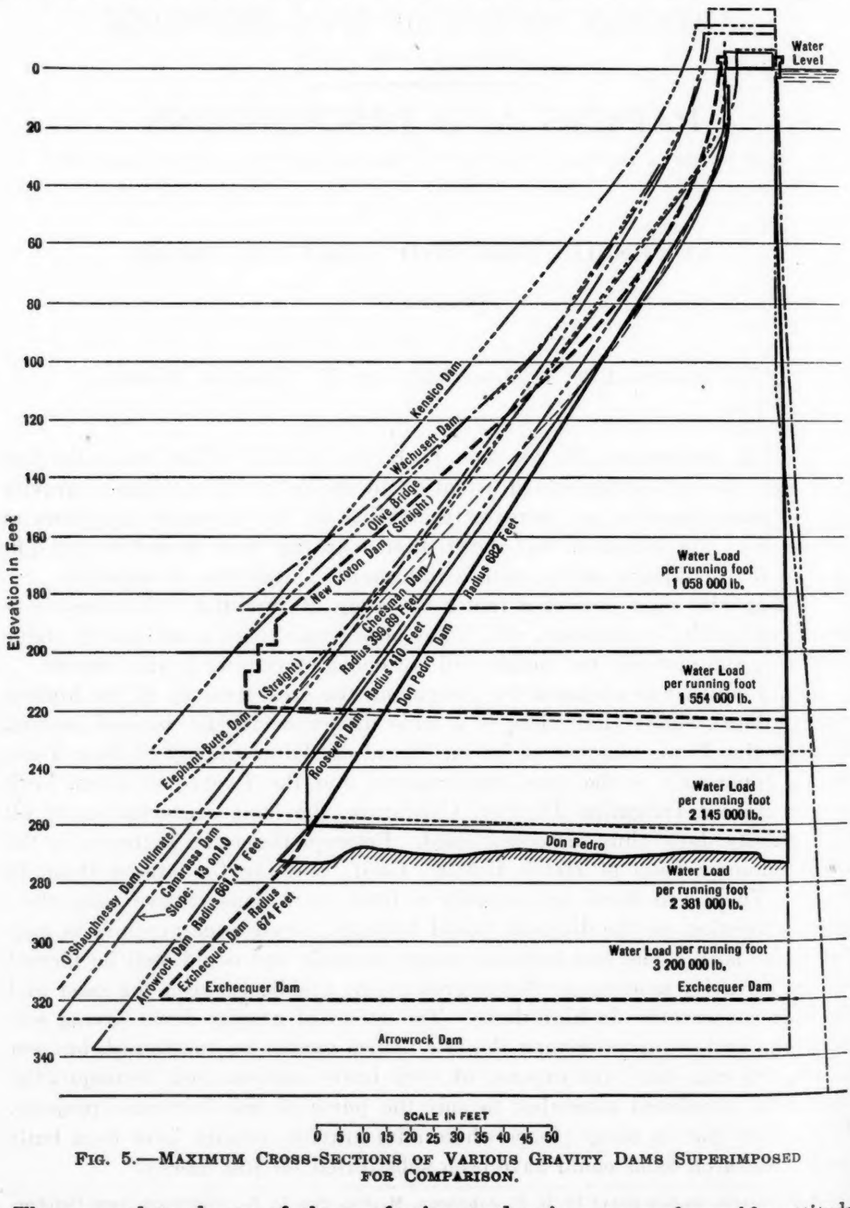

FIG. 5.—MAXIMUM CROSS-SECTIONS OF VARIOUS GRAVITY DAMS SUPERIMPOSED FOR COMPARISON.

The general tendency of the profession to-day is a more favorable attitude toward the arch type in places where arch dams will fit. The problem has become better understood. The merits of strength and economy inherent in

an arch dam cannot be overlooked any longer by even the most enthusiastic of gravity dam supporters.

The author's reference to the Exchequer Dam* is of interest to the writer who, at one time, was connected with that project and, therefore, is well acquainted with it. An arch dam on this site would have been cheaper and safer than the one built. At that time, the State (California) authorities' distrust in arch dams prevented the construction of the one shown in Fig. 6. This design was made by A. C. Hoff, Chief Designing Engineer of the Constant Angle Arch Dam Company, and the structure was nearly 20 ft. high when the so-called gravity arch dam, described by the author, was substituted. The arch dam (Fig. 6) required 310 000 cu. yd. of concrete as compared with 370 000 cu. yd. for the gravity dam, calculated to the same estimated rock contours. The excavation proved to be more than the estimated amount, but that would not have greatly disturbed the ratio of saving in material. Therefore, a more expensive dam was substituted for a cheaper one.

The design (Fig. 6) was made so that the stresses in the arch dam, using the Cain method of calculation and assuming the entire load to be carried by the arch down to and including Elevation 500, were about 500 lb. per sq. in., compression, and less than 100 lb., tension, which is considered conservative. At and below Elevation 475 the arch does not carry all the load. Shear and various other actions take the principal part on account of the arch being short and thick, and close to the bottom. If, for instance, all the load on the dam below Elevation 475 was transmitted to the foundation through shear action along the rock contact, the average shear would be 45 lb. per sq. in. As the punching shear cannot be transmitted without calling into play arch and cantilever action, a division of load takes place. Although it is hopeless to attempt to calculate the exact distribution, it is easily seen that the part of the load falling on the arch is only small and, therefore, will not cause dangerous tension anywhere in the faces. Horizontal tension along any of the faces of an arch is not dangerous if paired with a compression of moderate value along the opposite face, and is never as dangerous as horizontal tension in the up-stream face of a gravity dam. Furthermore, any apparent tension along the up-stream face will always be greatly relieved or entirely compensated by the swelling effect of the water on the concrete.

The arch at, and below, Elevation 475 will carry the full water load, but apparent tension of more than 100 lb. per sq. in. would develop, and, at present, this tension is regarded as the upper limit. Using the simple cylinder formula as a check, the average stress at Elevation 475 would be 235 lb. per sq. in., assuming all the load to be carried by the arch. Although the stresses are not uniformly distributed for normal load, they are practically thus distributed at the point of failure, which point, therefore, evidently will not be reached until the load has been increased more than ten times the normal at Elevation 475.

Vertical cantilever action in the ordinary sense has not been counted on to support any load inasmuch as under the most unfavorable condition of

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1141.

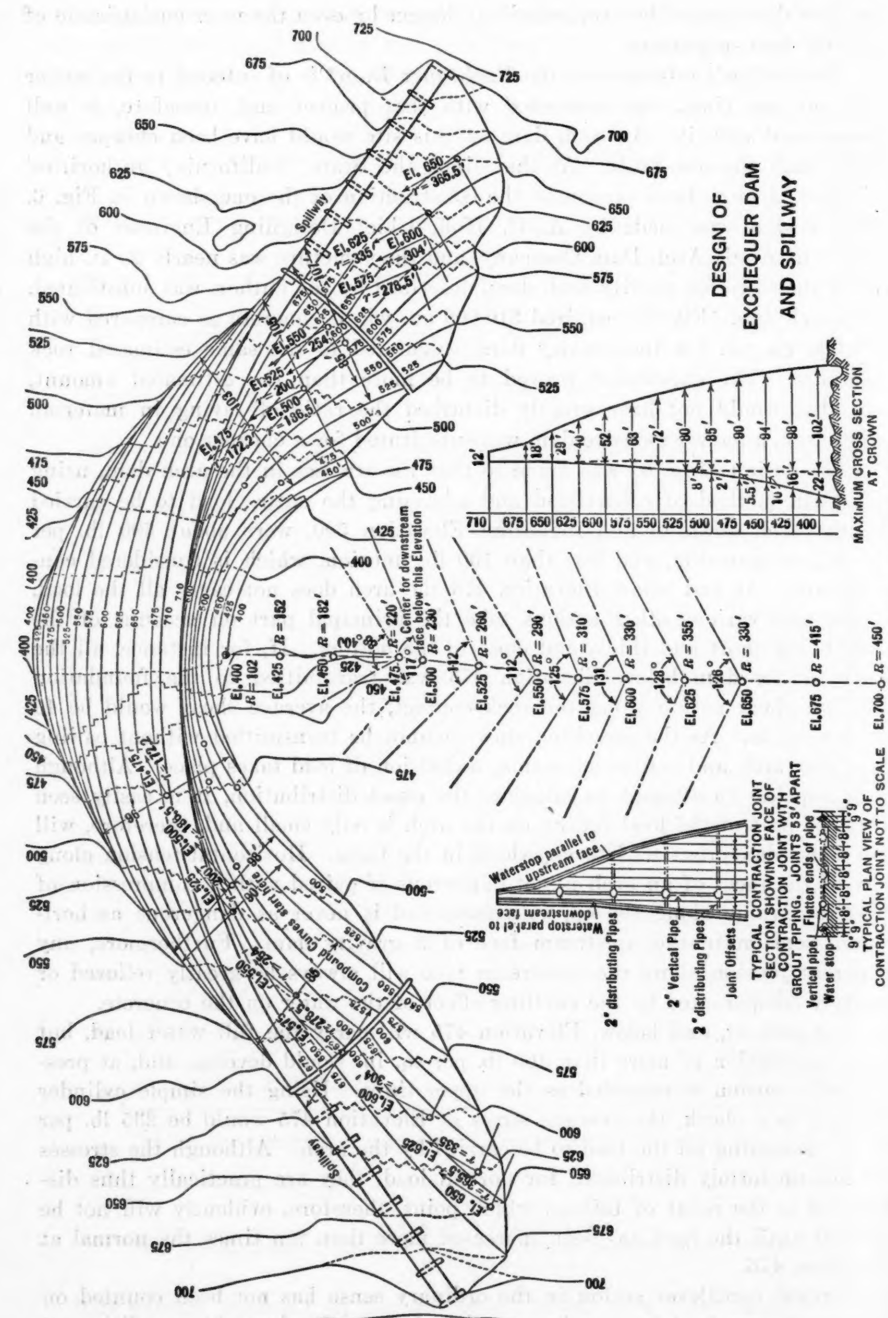

FIG. 6.—DESIGN OF EXCHEQUER DAM AND SPILLWAY.

water-soaking effect (a maximum water-soaking along the up-stream face decreasing gradually to nothing along the down-stream face), the cantilever bends in a down-stream direction without developing any resistance against the water pressure, and, therefore, it cannot carry any load. It is hardly possible to conceive that the water-soaking effect stops at the up-stream face, the remainder of the dam body being dry. Capillary action is always present to some extent.

Allowing for some faulty filling of the contraction joints, and for any conceivable disadvantageous action of the load, this dam can be relied on to possess a minimum factor of safety of five, and this is as much as is economically desirable.

The gravity dam which was actually built has a cross-section, as shown in Fig. 5, and is arched in plan. The arching of a gravity dam may look good, but otherwise it is a detriment to the stability of the cantilever as shown by the author. Unless the up-stream radius is much shorter than 674.7 ft., or the contraction joints are grouted properly, there will be no arch action to aid the cantilever with reservoir full during, and right after, cold weather. During this time the contraction joints have opened at the rate of approximately $\frac{1}{4}$ in. per 50 ft. length of crest and the voussoirs, therefore, have no contact.

One must not forget that, in arching a dam, the area on which the load acts becomes larger than that for a straight dam having the same amount of material and, at the same time, the center of gravity of the whole mass moves in a down-stream direction, both these items being detrimental to the stability of a dam acting by gravity.

A straight gravity dam of the same total mass as a curved one would have a greater cross-section; the distance between the center of gravity and the down-stream toe would also be greater. As the safety factor is in direct proportion to the product of weight and distance from the turning point, it is plain that it is uneconomical to arch a gravity dam in plan without having the contraction joints properly closed in order to obtain arch action.

Before arch action can take place with the voussoirs separated by a contraction crack, the middle of the dam must first slide bodily in a down-stream direction, but after such a slide it would be very difficult to make the structure water-tight. The Exchequer Dam, at the middle, should deflect slightly more than 1 in. down stream when fully loaded at the crest. This corresponds to a shortening of 0.852 in. at the crest of the dam.

Ordinarily, at the end of a cold period, the length of a dam decreases $\frac{1}{4}$ in. in 50 ft. The contraction joints open $\frac{1}{4}$ in., if spaced 50 ft. apart, cracks appear, or else tension is set up. During March and April, the crest of the Exchequer Dam, therefore, would have spaces between the voussoirs of approximately 4 in. with reservoir empty (the crest is more than 800 ft. long).

The cantilever deflection due to full load only shortened the crest 0.852 in., and there is, therefore, a space of approximately 3 in. to be closed with reservoir full before contact between the voussoirs is possible, and only after closing can arch action begin. This dam, therefore, must stand by virtue of

its weight, and due to the vertical tension along the up-stream face, the margin of safety is uncertain and cannot be very great.

It is not very conservative, to say the least, to allow any vertical tension along the up-stream face of a gravity dam. This is altogether different from having a certain amount of horizontal tension in an arch dam. If vertical tension causes a crack in a gravity structure, the dam is a failure; in an arch, a crack from horizontal tension would merely cause a change in the value of the compression. This is the main reason that horizontal tension in one or the other faces of an arch should not be compared with a high compression in the opposite face, as mentioned.

The Exchequer Dam sustains a higher unbalanced water pressure than any other dam, and it possesses the slimmest cross-section thus far attempted for high dams. It will not fail for it has a factor of safety greater than one if it withstands the tension in the up-stream face. If not, arch action will come into play after a slide of some part of the structure in a down-stream direction. This, however, would be a failure, although not a calamity.

This dam, however, was successful in competition with the arch dam shown in Fig. 6, although the arch required 60 000 yd. less material for its construction. A full discussion on the subject would be very desirable as much antagonism still seems to exist against arch dams and too much faith seems to be placed in almost any structure called a gravity dam.

P. WILHELM WERNER,* ASSOC. M. AM. SOC. C. E. (by letter).†—The author is evidently considering curved gravity dams with convex up-stream face. In this connection, however, it is interesting to compare the so-called arched gravity dam with a curved gravity dam presenting the rather unusual aspect

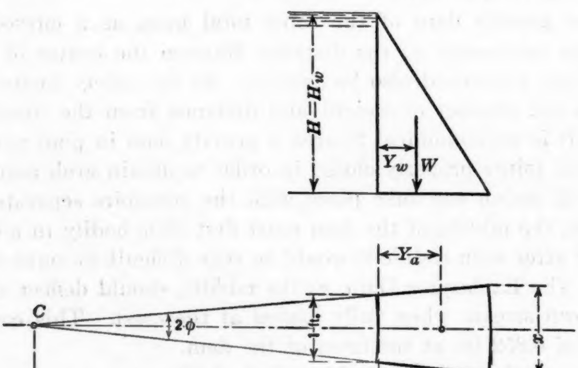

FIG. 7.—DAM WITH CONVEX DOWN-STREAM FACE.

of a convex down-stream face (Fig. 7). This type possesses some real advantages, in regard to stability, etc., that makes it worth while to consider in any actual case.

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* Designing Engr., Stockholm, Sweden.

† Received by the Secretary, September 6, 1927.

Using, as far as applicable, the same nomenclature and the same approximations for the computations as the author, the following set of formulas is developed:

$$y_a = \frac{B}{2} \cdot \frac{R_u - \frac{B}{3}}{R_u - \frac{B}{2}}$$

$$y_w = \frac{B}{3} \cdot \frac{R_u - \frac{B}{2}}{R_u - \frac{2}{3}B}$$

$$x = \frac{R_u}{R_u - B}$$

$$I = \frac{B^3}{36} \cdot \frac{1 + 4x + x^2}{1 + x}$$

$$A = \frac{B}{2}(1 + x)$$

$$W = \frac{B}{2} \cdot \frac{R_u - \frac{2}{3}B}{R_u - B} H w$$

$$M = -\frac{62.5}{6} \cdot H_w{}^3 + W(y_a - y_w)$$

Assume that the Exchequer Dam was designed with convex down-stream face, but of equal volume; so that, $H = H_w = 272.5$ ft.; $B = 171.4$ ft.; $R_u = 731.84$ ft.; $w = 160$ lb. per cu. ft.; with no uplift. This gives, $y_a = 89.5$ ft.; $y_w = 59.8$ ft.; $x = 1.305$ ft.; $I = 482\,000$ ft.^{4}; $A = 197.3$ sq. ft. $W = 4\,120\,000$ lb.; and $M = -88\,800\,000$ ft-lb. Thus, from the author's Equation (14),* the vertical stresses at the heel and toe, respectively, are 30.6 and 250 lb. per sq. in., both compression.

It is especially interesting to investigate the extent to which the base width of this dam can be reduced in order to obtain zero stress at the heel with reservoir full. If $B = 164$ ft., and the other dimensions are as given previously, the values are: $y_a = 85.5$ ft.; $y_w = 57.0$ ft.; $x = 1.29$ ft.; $I = 418\,000$ ft.^{4}; $A = 187.5$ sq. ft.; $W = 3\,920\,000$ lb.; and $M = -99\,100\,000$ ft-lb. Thus, from Equation (14), the vertical stress at the heel is 4.1 lb. per sq. in., compression; that is, practically zero.

Assuming that this section applies between two points, say, 200 ft. apart, the actual length of the dam would be about 201 ft. The volume then equals,

$$0.5 \times 272.5 \times 164 \times 201 = 4\,500\,000 \text{ cu. ft.}$$

The volume of a straight dam (base width = 171.4 ft.) between the same points would be,

$$0.5 \times 272.5 \times 171.4 \times 200 = 4\,670\,000 \text{ cu. ft.}$$

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1139.

It is seen that the curved dam with the convex down-stream face proves to be nearly 4% cheaper than a straight dam designed according to the same governing rule. It should be noted that the assumed curvature may, or may not, be the most favorable one. It seems certain, however, that this involves a mathematical problem of finding the least cost in each actual case.

The layout of a dam in plan is, of course, to a great extent, governed by other influencing factors than stability. The geological formation at the dam site, the desirability of increasing the discharging length of the crest, and making the flood water converge better into the natural river channel below, are such factors. Aside from these points, however, under certain conditions, some economy may evidently result in giving the dam a slightly convex down-stream face. The writer believes that the preference generally given to the arched gravity dam, is largely due to the additional support which the dam is assumed to receive from the arch. As arched gravity dams are usually constructed, however, this conception seems to be more or less illusory, although the arch action may prevent actual failure, if and when the cantilever has been broken. The writer is of the opinion that if it can be ascertained beyond doubt, that the dam receives some support from the arch under working conditions, this should be taken into consideration in the design in order to obtain an economic structure. In case arch action cannot be absolutely ascertained, however, it seems more consistent with a proper economic design to impose upon the dam some other condition that is more easily and clearly realized.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

RELATION OF ROAD TYPE TO TIRE WEAR

Discussion*

By Troy Carmichael, M. Am. Soc. C. E.

Troy Carmichael,† M. Am. Soc. C. E. (by letter).‡—Since 1919, there seems to have been an ever-growing tendency among engineers to resort to accelerated tests over a short period of time in order to prove the soundness of some new design or theory. In discussing this paper, two main points should be brought to the attention of members: First, the necessity of keeping tests free from the taint of commercialism; and, second, the design of a test to eliminate all variable factors as far as possible.

Engineers have the reputation of being technically trained men striving to inform the public, solely for its own benefit, which of several plans is the best one to adopt. This reputation is valuable, and members of the profession should resent any practice, whether intentional or unintentional, that will give the public reason to believe that they are being influenced by some private interest.

In Table 1,§ showing results of tests, it is seen that no tests were made on asphaltic types with Cars Nos. 1 and 2, these being the lightest cars; yet in Table 2,‖ showing a summary of the tests, the best results on concrete with Car No. 1 are given the most prominent position on the page. The average of all tests on concrete is shown, and below, the average on all tests on Bitulithic—the only higher type of asphaltic construction—is shown. This would naturally lead a casual observer to believe that the tire wear on concrete pavements amounted to 0.0852 lb. per 1 000 miles, and that on a Bitulithic

* This discussion (of the paper by O. L. Waller and H. E. Phelps, Members, Am. Soc. C. E., presented at the meeting of the Highway Division, Seattle, Wash., July 15, 1926, and published in August, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Vancouver, B. C., Canada.

‡ Received by the Secretary, December 10, 1926.

§ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1193.

‖ *Loc. cit.*, p. 1194.

pavement the average would be 0.1247 lb. It would take a careful study of this paper to discover that no light cars were used on the asphaltic pavements. This is grossly unfair. An average result of tire wear for two or more cars of different weights means nothing.

In Table 3* the unfairness is even more apparent because the averages of all tests on each type of pavement are shown together. From these figures an index number is determined which is given to the profession without any qualifying statement. It is easy enough to see that the index number for asphaltic pavements would be materially lowered had the tests been truly comparative. The last line in Table 3 gives the average of the seven best tests for concrete, but no attempt is made to give the asphalt pavement the same publicity. Another unfair discrimination in favor of Portland cement concrete is in the fact that these pavements were either new or practically new, long stretches being less than a year old. The Bitulithic, while in good condition, was 10 years old. A comparative tire-wear test with this piece of pavement and a Portland cement concrete pavement 10 years old, would be of real interest to the engineers responsible for the design of public highways and streets.

Asphaltic concrete made from coarse aggregate presents the roughest surface to the tire of any mixed asphaltic pavement. Had an asphaltic concrete, fine-aggregate type (so-called Topeka mix) or sheet asphalt been chosen, their linoleum-like surfaces would have given a far smaller index number for tire wear than was obtained. Such discrepancies, although they seem trivial, gain importance from the fact that they have been abused by materials companies. Wide propaganda based on these very data has been issued.

These are just a few of the more glaring instances where the authors' methods have allowed their experiment station to become an advertising medium for private interests. Numerous others could be pointed out.

The other main question concerns the design of a test to eliminate all variable factors. A tire-wear test of a few hundred miles here and there with different climatic conditions, types of car, and other "variables" and "personal equations", too numerous to mention, is of no scientific value. Apparently, Professor Waller has arrived at that conclusion. In addition to the doubts in the "Introduction"† and concluding pages, practically every section opens with such frank statements as:

"The few tests of tire wear so far made at different speeds reveal no concurrent relationship between these factors."

* * * * * *

"Various runs with the same car did not show closely similar tire wear."

* * * * * *

"Many additional data must be secured before dependable conclusions may be reached."

These statements would seem to indicate that the results obtained from these tests have no real scientific value, and yet the authors have hit on a subject that is of considerable monetary importance to automobile users.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1194.

† *Loc. cit.*, p. 1190, *et seq.*

The annual tire bill in the United States amounts to approximately \$800 000 000, and the difference between the two highest types tested (concrete and asphalt), amounts to \$50 000 000 or \$60 000 000 per year in favor of asphalt.

Therefore, it appears that this test is important, and that a relative tire-wear index number for the various types of roadways prevalent in this country would be of value to the profession, and certainly very interesting. It would seem that a very reliable figure could be obtained under the following conditions:

(*a*) Testing to destruction instead of by trips of a few hundred miles.
(*b*) Using parallel or adjoining roads as long as available under practically the same climatic, grade, and age conditions.
(*c*) Selecting road surfaces as nearly perfect as possible and five years old or more.
(*d*) Keeping uniform speed as near as possible to 30 miles per hour and 300 miles per calendar day.
(*e*) Making tests simultaneously, so that weather, heat, and moisture conditions are absolutely identical.
(*f*) Starting tests with the same make of new tires and practically new cars of the same make and model.

As Professor Waller suggests, nothing short of such a test will give "accurately, final relative results".

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

STREAM POLLUTION IN THE PACIFIC NORTHWEST
Discussion*

By Messrs. Augustus Smith, and A. H. Dimock.

Augustus Smith,† M. Am. Soc. C. E.—It may be of interest to consider what is being done in New Jersey relative to stream pollution. The State has been handling the matter of stream pollution through the State Department of Health. It has also a Fish and Game Commission, which has been very active. Frequently, this Commission has come into contact with the other State Departments because it could not get all the support it wanted in its work of preventing the pollution of streams.

In 1924, the State created the Sanitary and Economic Water Commission, of which the speaker is a member. The idea behind that legislation was to create "a permanent jury", so to speak, to give judgments as to the "economic" policy on the pollution of any stream or water in the State. The new Commission was formed by the Governor appointing one member from the State Department of Health, one from the Fish and Game Commission, one from the Shell Fishery Commission, one from the State Board of Conservation and Development (which has charge of all State lands), and one from the Board of Commerce and Navigation (which deals with problems concerning navigable waters), and the Attorney General of the State.

The Commission has been functioning only for about a year, but one of its first acts was to adopt the method of classifying streams recommended by W. L. Stevenson, M. Am. Soc. C. E. He divided streams into three classes:

(*a*) Those streams, or portions of streams, used or reserved for potable purposes.

(*b*) Those streams clean enough for recreation, for enhancing the value of real estate, and for fishing.

(*c*) Those streams passing through communities where the industrial needs are paramount.

* Discussion of the paper by the late William F. Allison, M. Am. Soc. C. E., continued from October, 1927, *Proceedings*.

† Pres., Bergen Point Iron Works, Bayonne, N. J.

It has seemed to this Commission, that the first step in trying to recover or preserve the purity of the streams, is to arrive at a general consensus of public opinion as to the best use of a stream from an economic standpoint.

A. H. DIMOCK,* M. AM. SOC. C. E.—After discussing various ways by which the waters of Puget Sound are polluted, the author comes to the general conclusion that all such polluting matter should be treated and sterilized before discharge. While the speaker is heartily in favor of keeping all waters in the best possible condition, and especially all fresh waters, nevertheless this conclusion seems somewhat sweeping.

Puget Sound, including Admiralty Inlet, is a body of water about 60 miles long from the Straits of Juan de Fuca to Tacoma, Wash., and from 3 to 6 miles wide. South of Tacoma it narrows to less than a mile and then widens and breaks up into an intricate maze of channels and bays. In this southern portion are situated a number of oyster beds. The bays are shallow, the flat areas extensive, and the movement of water is more or less restricted. The treatment of sewage entering this portion becomes, in general, more desirable. Each particular problem, however, must be decided in the light of its own local conditions.

In the larger part of Puget Sound, north from Tacoma, however, the channel is very deep, varying as much as 120 fathoms. The tides have a range of 16 ft., and there is necessarily a considerable movement of the water. The volume and character of these movements are not definitely known. Oyster beds are few. There is none along the main channel, and the shores, generally, have good slopes. It would seem, therefore, that the oxidizing capacity of this part of Puget Sound is an asset of considerable importance to the towns and cities on its banks, and may be utilized without detriment to health, the public welfare, or private business.

When the writer was designing the main sewerage system for the larger part of Seattle, Wash., about twenty years ago, extensive observations of currents were made by floats. This survey revealed many beaches which were highly undesirable as points of discharge for sewage, for the reason that the currents set chiefly toward the shore. Some places were found, however, where the currents for 80% of the time were offshore. The outfall of a main sewer 12 ft. in diameter was located at such a place and extended to a point where the water was 40 ft. deep. The only trouble that has been experienced at this point, has been at outfalls where discharge was permitted on the surface or at shallow depths. The remedy is obvious. It would certainly be a "counsel of perfection", under such conditions, to spend large sums for artificial treatment plants, when Nature has provided one far more efficient, the use of which is free of operating charges and involves no menace to health.

The case of Lake Washington, in Seattle, is different. This lake is 20 miles long by 2 to 4 miles wide and lies parallel to the salt-water front at a distance of 2 to 7 miles. Its shore, within the city limits, is used almost exclusively for residential purposes, with the exception of a few ferry landings, one saw-

* Cons. Engr., Seattle, Wash.

mill, and other minor business. The lake is subject to navigation, but this use is small. The sewage from as much of the drainage area as was possible has been intercepted and carried to the salt-water outfalls; but there remained about 4 500 acres, the sewage from which has been discharged into the lake. The growth of population and the largely increased use of bathing beaches created the necessity for some action. It was proposed to build a series of Imhoff tanks and to chlorinate the effluent. The speaker investigated the problem on behalf of the City and came to the general conclusion that Lake Washington was an asset of great value to the city both because of the desirability of the residential sites along its shores, with their entrancing views of lake and forests and snow-crowned mountains, and because of its recreational possibilities; a use which will be more and more essential to the public welfare as the population increases. For these reasons it was held desirable to exclude from its shores everything that was even suggestive of unpleasant things, and that sewage treatment plants, which would necessarily be more or less visible, and from which odors might emanate, should not be permitted if some other means could be found within a reasonable cost limit.

The speaker sought to determine a standard of purity for the waters of Lake Washington. It is not used as a source of water supply except in the sparsely settled section on the east shore, several miles from the Seattle side. Its main use is recreational. In 1925, 365 000 people used the bathing beaches, which are far more popular than the salt-water beaches due to the higher temperature of the water and the more convenient access. A considerable diversity of opinion as to a proper standard was found to exist among the experts, ranging from not more than 10 *B. Coli* per cu. cm. in swimming pools in New York and in salt-water beaches in California to a drinking-water standard insisted on by some authorities. Just how the latter standard can be maintained either at an open bathing beach or in a closed swimming pool is difficult to understand. The chief danger is from infection arising from promiscuous bathing at the same time of diseased and healthy persons. No practicable excess of chlorine can prevent, although it may reduce, this danger. The remedy would seem to lie in proper inspection and regulation and not in the futile endeavor to maintain a drinking-water standard. It was the speaker's conclusion, that, if the water of Lake Washington was negative as to *B. Coli* in 1-cu. cm. samples, it would be safe anywhere for individual bathing, and that such a standard might be attained by diverting all sewage. At the bathing beaches, because of the pollution contributed by the bathers themselves, it was thought that regulation of use and chlorination would be essential, even if the water in the lake could be brought to the drinking-water standard.

The means recommended to accomplish the desired results were the diversion from the lake of all sewage plus rain water to a total amount of 0.033 sec-ft. per acre, by the construction in one case of a tunnel and in others of suitable automatic pumping stations discharging into existing intercepting sewers. These stations are to be provided with storm-water overflows discharging in about 30 ft of water. It is thought that under the conditions that obtain, no further treatment of storm water will be necessary. The average

annual rainfall in Seattle is 33 in. The intensity is remarkably low. The number of storms equal to or exceeding the rates shown during a period of 30 years are, as follows:

0.80 in. per hour........	0	0.40 in. per hour........	12
0.70 " " "	1	0.36 " " "	14
0.60 " " "	2	0.30 " " "	30
0.50 " " "	8	0.25 " " "	55

Using a capacity of 0.033 sec-ft. per acre as stated, and assuming a population of 25 per acre and a run-off factor of 0.25, it was found that all rains of less than 0.12 in. per hour would be diverted from the lake and that there would be only 28 hours per year when that rate would be exceeded; an average of 1 hour's rain every 13 days.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

BASIC INFORMATION NEEDED FOR A REGIONAL PLAN

Discussion*

By Messrs. Russell V. Black, George F. Unger, Arthur A. Shurtleff, Hale J. Walker, Joseph W. Shirley, Guy Wilfrid Hayler, William Bowie, and Howard Strong.

Russell V. Black,† Assoc. M. Am. Soc. C. E.—In formulating a survey program for the Philadelphia Tri-State District regional study many diverse points of view have been expressed. One man of professional prominence advises, "Stop gathering information. Make plans." Another, of equal prominence in his field, remarks on the superficial character of the survey program of the New York Regional Plan, which has spent nearly five years and approximately $750 000, largely in gathering data and getting acquainted with its region.

The Philadelphia District is operating on very limited funds. In the light of planning experience and personal judgment, its members cannot proceed to plan without first learning a great deal more than is now known about their region. On the other hand, they can afford neither time nor money for the collection of unnecessary data, and depending, as they do, on public support, they must show some planning progress comparatively early in the work.

They have looked forward with a great deal of anticipation to receiving suggestions from Mr. Lewis' paper and the attendant discussions, in the hope of obtaining some criterion of selection whereby they might eliminate certain classes of more or less needless data and concentrate on certain others agreed to be important.

Mr. Lewis has given much helpful guidance as to the extent of material to be obtained and as to survey and study procedure; but the speaker is disappointed in not receiving more specific suggestions as to basic information

* This discussion (of the paper by Harold M. Lewis, M. Am. Soc. C. E., presented at the meeting of the City Planning Division, Philadelphia, Pa., October 7, 1926, and published in September, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Planning Engr., Regional Planning Federation, Philadelphia Tri-State Dist., Philadelphia, Pa.

indispensable to the preparation of a workable regional plan. It may be that the field of regional planning is still too little explored and the problems of the many districts too diverse to offer any very definite general suggestions of scientific approach. However, it would seem that, compelled as they have been to blaze absolutely new trails of survey and research, the several regional planning projects which have made noteworthy progress, must have made some recognizable errors; must have gone up some "blind alleys"; and that, were they to do the work over again, they would go about it somewhat differently. It is this experience which the Philadelphia District would find so valuable and which it has hoped might, in some way, be made available.

From brief experience, two or three conclusions in this matter of survey making have become evident. In the natural development of regional planning from city planning, it is questionable, as Mr. Lewis has described it, if the tendency to "follow the leader" has not led many planners to collect a great deal of thoroughly useless information. In the early and quite experimental age of city planning, men, new to the field, groped about for a formula and some sort of foundation with, and upon, which to erect their future cities. They conceived that certain groups of information might be valuable, and formulated a survey program accordingly. There is some evidence that present methods have not advanced far beyond those first efforts, and that surveys still hold a little of the old chaff that could well be discarded.

The regional plan should not concern itself with matters of purely local character. Local administrations are naturally somewhat jealous of their prerogatives and can usually handle their affairs to better advantage than any outside body. The plan should be limited in its application to those problems and development projects which involve two or more political jurisdictions, and for the solution and conduct of which no other machinery exists. A Regional Planning Federation, as an agency for the promotion of sound planning, might encourage the proper provision and distribution of neighborhood playgrounds and school grounds; but matters of such local moment should not be made a part of the regional plan. Likewise, it might be a function of a Federation, in furthering the general welfare of the region, to encourage a proper alignment of minor streets, a good width of lot, or the proper distribution of neighborhood store groups; but, again, these things are not a part of a regional plan. Such a plan involves determining the broad uses of land, the completion of a system of connecting and trunk highways, regional and intercommunity parks, regional water supply, the purification of streams, the conservation of natural resources, the broader phases of transportation, co-operative sewage and drainage, and other matters demanding the joint effort of two or more political sub-divisions. It follows that physical data, collected to serve as basis for a regional plan, should be in scale with its purpose, and that, by keeping this scale constantly in mind, a great deal of useless material can be eliminated.

In the field of economic and social survey it is more difficult to know where to stop searching. Too much cannot be known about the laws that govern the conduct of the human race nor of the foundation and controlling

forces of an ever-changing economic system. At this moment, every one is pitifully ignorant of the degree to which the modern great city is inherently fundamental to the nature and progress of mankind and the degree to which it is simply a diseased overgrowth. Until more is known certainly of what this phenomenon of city growth is, decentralization of population cannot be adjudged as even desirable; much less can it be attained through present feeble efforts. This is the sort of knowledge needed in order to direct the destinies of great metropolitan areas; but it is the kind of knowledge that requires much more painstaking research than present-day planning organizations can afford in time or money. Fortunately, the knowledge gained in these directions in one part of the country, and to some extent, the world, is almost equally applicable in every other part. Supplemented by a familiarity with the local social and political structure, the general background to be obtained from present-day observation would probably provide as good a sociological basis for regional planning procedure as can be found.

In developing a program, local regional planning has been divided, somewhat roughly, into two steps: Step (1), preliminary, getting acquainted with the region, making tentative suggestions as need may demand, and feeling out and developing public opinion; and Step (2), laying down the master plan, piece by piece, and as based on an exact knowledge of existing conditions and present and probable future needs.

Step (1) is usually attended by small funds and large public skepticism. It must be conducted, not only with the idea of making sound progress, but equally with the object of gaining public attention and approval. A great deal of information, of little purpose other than to awaken public interest, must be collected and appealingly presented. This period is largely educational, with first efforts bent toward pointing out general existing conditions; the sorry situation resulting from lack of earlier planning and the possibilities of planning for the future. The material collected for Step (1), while necessarily insufficient as a basis for definite planning proposals, should, nevertheless, be such as to serve as a foundation for the more detailed information to follow.

This refers, largely, to physical surveys which might include the topography of land, as given by the U. S. Geological Survey; the distribution and relative density of population, defined by minor civil divisions for several preceding census periods; the larger existing parks and playgrounds, cemeteries, and other potential open places; principal waterways; steam railways, and electric interurban lines; aviation fields; general sewage disposal facilities and character of water supply; extent of public service—gas, electricity, and telephones; other similar classes of data; and, perhaps most important, the broad use of land, including indication of the forest areas, farm lands, open suburban development, sub-divisions not built upon, principal business and industrial areas, large parks, private recreation areas, and large institution grounds.

This should provide sufficient basis for a fairly comprehensive understanding and statement of the condition and needs of the region. Coupled with a reasonable familiarity with the social and economic status of the region and

with an appreciation of the bearing of existing legislation, this much of a survey should permit of tentative planning proposals and the first roughing out of the plan. What the schedule of procedure in making these tentative proposals might be, and whether or not the preliminary survey, as outlined, is to be completed before any of the planning studies are undertaken, depends on local necessity. It would seem, however, that at least this much of a picture of the region should be finished before very much planning is done.

Step (2) would involve more specific recommendations and the laying down of a workable skeleton plan with a considerable degree of exactitude. The gaining of public support is no less important at this stage of the work because, without the sympathy and co-operation of public officials and the sanction of the taxpayers, nothing can be accomplished; but the engineering and planning factors must be given more thorough consideration. There should be available at this phase of the study an aero-photographic map of the entire region at a scale not smaller than 2 000 ft. to the inch. There should be an accurate topographic map, in a scale ranging from 1 000 ft. to 200 ft. to the inch, depending on the nature and probable intensity of development of the locality. Each factor of the plan, such as sewage disposal and drainage; water supply; rail, water, and air transportation; highways and bridges; and parks and other recreational areas, would require its special consideration, although closely co-ordinated, detailed, and studied. Perhaps this phase of regional planning should ultimately be directed into the hands of a permanent official or semi-official body, functioning to execute the plan and to keep it adjusted to changing needs.

Such a program (especially Step (2)) is costly, but regional planning is an important undertaking which, if effective, must guide billions of dollars worth of public and private development projects. It holds the possibility of saving many times any possible engineering or planning costs. Done wisely, no other single instrument offers as large an opportunity for the advancement of the general welfare. Ill-considered and superficially done regional planning holds almost equal opportunity for injury. It deserves, and must ultimately have, all needed public funds and the best engineering, economic, and sociologic thought of the time.

George F. Unger,* Assoc. M. Am. Soc. C. E. (by letter).†—It is of great interest and value to have a paper presented on basic information needed for a regional plan. Many cities are growing so rapidly that they are over-reaching their present boundaries, thereby creating a decided influence on the adjacent territory and extending activities to various political sub-divisions, all of which have a common interest. This necessarily involves a treatment of the larger area in the broad sense of regional planning, and when presented to the planners of a metropolitan district, it immediately suggests the formulation of some definite outline of procedure.

The Niagara Frontier Planning Board is significant in that it was created in a way differing somewhat from other regional planning bodies. Legis-

* Chf. Engr., Niagara Frontier Planning Board, Tonawanda, N. Y.

† Received by the Secretary, October 16, 1926.

lation was obtained, giving it an official status and providing for a method of financing with public funds, the District included being the political subdivisions of Erie and Niagara Counties. This seemed to be the territory containing communities whose interests are common and whose commercial and recreational life is closely allied with the principal business centers. Subsequent to this legislation, funds were appropriated by the respective Boards of Supervisors, for carrying on the work. The Board consists of twelve *ex officio* members, designated by law, including the Mayors of the six cities of Erie and Niagara Counties and three members of the Boards of Supervisors of each county. The thirteenth member is chosen by the Board and acts as its Chairman.

The area which constitutes this District is about 1 556 sq. miles, extending from Lake Ontario on the north, to Cattaraugus Creek on the south, the Niagara River and Lake Erie on the west, and the county lines on the east, covering a distance of 60 miles north and south and 25 miles east and west, and containing 6 cities, 22 villages, and 37 townships.

Several commercial and industrial centers are located within this District, Buffalo being the largest, with Niagara Falls next, at a distance of 18 miles northwest. Tonawanda and North Tonawanda lie midway between, with Lockport in the northeastern section and Lackawanna bordering on the southern boundary of Buffalo. A portion of Canada contiguous to the Niagara River and lying to the west is also considered in close relationship to this area and planning is being developed to connect it with proposed structures crossing the Niagara River.

The District is divided, by the topography of the land, into four distinct natural sub-divisions: First, the table-land at the north, bordering on Lake Ontario, and extending with a gradual rise in elevation to the Lewiston Escarpment. This section is known as the "Niagara Fruit Belt," being devoted largely to fruit growing, but with extensive general farming. At the escarpment, the elevation rises rather abruptly from an average of 400 ft. above sea level on the lower table-land to another elevation of 600 ft., running gradually to the south over a considerable area about 800 ft. In this second section are the six thriving cities and a network of railroads, highways, and navigable waters, and within which the tremendous development of the region is largely confined. The third division comprises rolling uplands with elevations from 800 to 1 200 ft., devoted to the dairy industry and general farming; while the fourth division, the extreme southern portion of the District, is fairly mountainous in character, rising to an elevation of 1 900 ft. above sea level. Although some portions of this hilly country are not easily accessible, it contributes liberally to the food supply of the District and also affords recreational facilities, forest preserves, and natural scenery.

While the outline made for obtaining data in this District does not conform exactly with that presented by Mr. Lewis, it carries the same general provisions. It is essential, also, to determine how the information that is collected is to be used in preparing a regional plan.

An accurate base map was first prepared on a scale of 1 in. to the mile, made from the U. S. Geological Survey Sheets, showing highways, railroads,

waterways, parks, boundaries of political sub-divisions, and contours at 100-ft. intervals. The size of this map is 42 by 72 in. Base maps of the District were also prepared on a scale of 1 in. equals 2 miles and 1 in. equals 4 miles. Both these scales were used for mapping statistical data in relation to population, valuations, highways, transportation, and traffic. Maps of the cities and villages were prepared on a scale of 1 in. equals 600 ft. and of townships on a scale of 1 in. equals 2 000 ft., these being used for the purpose of more detailed studies in regard to the various communities.

The function of this Planning Board is not only to study the District as a whole, but to devote considerable attention to community planning as well, organizing local planning bodies to carry out the work of their own political sub-division. Due to the fact that this District has only been organized for about a year, most of the time has been spent on the physical survey, mapping the data in regard to topography, boundaries, population, highways, transportation facilities, parks and parkways, water supply, and sewage disposal. This information has been assembled in such a manner and extent as to enable some definite recommendations to be made on problems that have already been developed and require immediate consideration. Detailed plans have been prepared for the purpose of making some of the necessary developments.

Because of the National and International reputation of this District, visited by more than 1 000 000 tourists every year, great importance is placed on the scenic wonders as well as the other available natural resources which include water power and waterway transportation.

One of the most noteworthy projects recently planned, is that of a Regional Parkway System, which will ultimately be developed along the Niagara River. There is located in the Niagara River an island of 28 sq. miles, called Grand Island, lying adjacent to the most rapidly developing section of the District. A straight line drawn from the center of Buffalo to the center of Niagara Falls, passes through the middle of this large undeveloped area. It seems very desirable to connect this area with the mainland. The construction of two bridges and a highway, crossing diagonally across Grand Island, will shorten the distance between Buffalo and Niagara Falls about 4 miles. It will then be possible to develop a parkway, about 6 miles long, on the western side of the island along the Niagara River. This parkway can be made to connect with the New York State Reservation at Niagara, and the parkway system laid out in and adjacent to Buffalo, thereby forming not only a link in the State Parkway System, but affording a beautiful driveway between the two largest communities. It will ultimately be extended northward along the Niagara River from Niagara Falls to Lake Ontario and southward through Erie County, passing to the east of Buffalo and connecting with Allegany and Letchworth State Parks.

Another project of regional importance recently completed is a highway bridge known as the "Peace Bridge," across the Niagara River at Buffalo to Fort Erie, Ont., Canada. The American terminus of this bridge is in one of the large city water-front parks, which is connected by parkways with the Buffalo Park System, while the Canadian terminus joins a parkway about 25 miles in length, extending northward along the shore of Niagara River to

Niagara Falls and Queenstown, where bridges now cross to the American side. The New York State Parkway previously mentioned, is planned so as to make a river drive from Lewiston to Buffalo, comparable with the improvement so admirably carried out by the Park Commission in Canada.

It is impossible in this brief discussion, to elaborate on the work done either in preparing for planning this Region, or the inestimable value of the data already obtained. As the work of gathering the necessary information proceeds, it will be found that the paper presented by Mr. Lewis, will be a guide in obtaining results more efficiently than heretofore, and that other regions will profit greatly by the experience of the planners for New York and Its Environs.

Arthur A. Shurtleff,* Esq. (by letter).†—This paper is sufficiently complete to be considered a good example of the kind of data which the engineer of to-day should collect for a great metropolitan area. Some data which at present are considered novel, will become, in later years, a matter of routine. Water supply and sewage disposal were novelties seventy-five years ago. Electric service was a novelty recently, and good motor traffic accommodation is a novelty now.

The writer hazards an opinion regarding one of the new problems that city planners will be forced to consider during the next two or three decades. By that time, it is to be hoped, the accommodation of vehicular transportation will have become fairly commonplace. Architectural excellence undoubtedly will be more general then; and better living and working conditions may be expected. Standardization of these more perfect things, however, will tend to create uniformity, which, on a vast scale, will become objectionable in itself.

Objectionable uniformity is something more than that which merely bores. It is a uniformity which becomes actively oppressive when repeated endlessly, mile after mile, no matter how complete or how attractive the objects may be in themselves. Beginnings of depressing uniformity of excellent things are to be seen in the central portion of the City of New York, and of less excellent things in the Everett-Chelsea-Somerville Districts of Boston, Mass. These depressing uniformities hurt the pleasantness of cities and, to that extent, threaten the continued earning power of the vast sums of money invested in great metropolitan regions, and they hurt the spirit of men.

The era is passing when a great city can hold out unique attractions of a compelling kind to the dweller in country districts. There was a time when great cities were maintained, in spite of heavy death rates, by the influx of families and workers from villages and farms seeking the city on account of its novelties, shelter, culture, entertainments, and its opportunities for social intercourse and employment. To-day, that situation is slowly tending to reverse. The urge from the great cities toward the suburbs and toward the villages is becoming stronger with the development of cheap transportation, with the growth of large isolated industrial plants, with the general provision of schools and hospitals, and with the facilities for ready construc-

* Landscape Archt., Boston, Mass.

† Received by the Secretary, October 16, 1926.

tion of homes equipped with "modern improvements" of extraordinary completeness. Opportunities for recreation in natural surroundings are also acting as a strong urge toward the towns.

Parks have been built in cities to provide an opportunity to enjoy recreation in rural surroundings. However, parks in great metropolitan cities are gradually assuming a new and exceedingly valuable rôle quite aside from recreation spaces. The mere existence of areas of open ground covered with trees and grass is becoming an enormous asset in breaking up the monotony of great city areas. In most metropolitan cities, strips of vacant private land still remain between the smaller integral cities and towns and give these centers sufficient local separation to save their individuality from complete eclipse in the great fabric of buildings and streets. When town after town becomes closely knitted to its neighbor, as, for example, the Dorchester-Roxbury District, of Boston, and the Brooklyn region, of Greater New York, the entity of villages and towns becomes lost. The resulting monotonous repetition of street upon street and building upon building appals the human spirit and cannot be ameliorated by perfection of transportation, excellence of architecture, street trees, or perfect housing and attractive employment, because each of these good things tends toward its own endless repetitions.

The time is fast approaching when metropolitan regions must include in their budgets monies for the maintenance of large areas of open ground distributed in long strips or winding bands throughout the built-up area to relieve monotony of this kind, quite aside from the recreational value which the ground or the landscapes, contained in such strips, may possess. Vast appropriations for metropolitan thoroughfares are common. Expenditures as great, or greater, for the acquisition of open spaces required to safeguard metropolitan areas from depreciation in value, through an intolerable increase in monotony resulting from too continuous up-building, are destined to become as common.

In coming years engineers will devote much time to the mapping and classification of vacant lands on the outskirts of cities and towns which are being absorbed in metropolitan districts. Much study will be needed to decide what land shall be saved for permanent strips or bands of open ground; its extent; its relation to the topography of hills and stream courses and lowlands; its contact with adjacent boundary streets and transportation routes; and the most favorable time for its acquisition. Evidently, these strips must be wider, or closer together, in cities which lack varied topography than in those the sites of which are naturally diversified.

The purpose of these costly public acquisitions will be the same as that for which even greater sums are being spent in private enterprises for the mere embellishment of office buildings, stores, factories, and residences; namely, to secure attractive appearance. In these times, not to secure attractive appearance in great private undertakings, means the risk of jeopardizing all the money invested. Mere structural strength, convenience, and comfort are not enough to meet the approval of the public.

The same approval must be reckoned with in great metropolitan areas. Mere convenience of transportation, beauty of buildings, excellence of light

and air conditions, absence of confusion in building heights and uses, opportunities for amusement, recreation, and employment, will not be enough to satisfy the man who is enmeshed too completely by walls and streets. He will demand freedom from depressing monotony, or he will go where he can find it, readily and cheaply, in the small city or town the call of which is becoming clearer and stronger each decade.

Metropolitan areas need the most intelligent study and watchfulness which men can apply to them to prevent this call from becoming destructive in its consequences, either to the great treasure of money and culture which is contained in them or to the human beings, counted by millions, who depend on the city as the environment of body and spirit.

HALE J. WALKER,* ESQ.—In looking over this paper one is impressed by the scope of the outline. Every kind of necessary survey information would certainly find its place somewhere in the various and broad sub-heads.

A definition of "open spaces"† would probably be as follows: All types of land, public, semi-public, and private, for the most part not built up in the third dimension, which gives to the region permanent breathing spaces. They have been described by some city planners as the "lungs of the city". Under "open spaces", information should be gathered, giving the location of all public lands, such as parks, parkways, and recreational areas, institutions (Government, State, County and City), town forests, aeroplane landings, waterways (rivers, lakes, ponds, creeks, swamps, reservoirs, and their catchment areas), and the location of public utilities requiring open space.

In gathering survey material the tendency has been to collect information on all public open spaces and to neglect semi-public and private open spaces.

Included under this heading then, will be information giving the location of semi-public and private open spaces that promise permanence, such as private schools and colleges, hospitals, sanitariums, cemeteries, churches, institutions, golf courses and country clubs, boat clubs, aeroplane landings, fair grounds, historic monuments and sites, commercial amusement parks, professional baseball fields, and swimming pools. There should also be noted that region of large country estates which could be made an effective open space by zoning. Some of the larger industries also furnish this type of open space. A large acreage is often required by gasoline and oil storage companies, furnishing an open space of sorts, but still a factor in the calculation of population density and in determining the character of future built-up areas. Information collected under the other sub-heads may also furnish material for open spaces—existing or proposed.

The topographical plan shows the basic physical features, such as mountains, lakes, forests, and other areas with strong topographical characteristics difficult of changing by the hand of man and particularly adaptable for park use. The mapping of population distribution also bears a direct relation to this subject. There might be added under this heading a collection of data on soil conditions and the mapping of existing agricultural lands. Agri-

* Town and City Planner, Cambridge, Mass.

† *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1507.

cultural belts and wedges have been recommended by city planners as breathing spaces for a large city or metropolitan region. Perhaps since, at present, city planners have no power to zone land to agricultural use, this type of open space may not seem very permanent. Nevertheless, the planner, in some cases, should consider land particularly adaptable for agricultural purposes as an open space, and design his region so as to discourage sub-division and to encourage truck gardening and agricultural pursuits. In designing agricultural land for this purpose, perhaps more intensive development of the area can be held off until it can be legally adopted for agricultural use.

The question of control of private open spaces makes the use of this information a somewhat hazardous problem in the design of a region. Of the list of private open spaces, colleges and cemeteries seem to be the most permanent. The first might fall under Mr. Lewis' heading,* "Recreation." The latter holds a doubtful position. It might be considered as a park, but properly it falls under the heading of sanitation. There are instances in California where cemeteries have been re-designed for park purposes. A new type of cemetery is also being designed with park drives, gravestones and monuments being prohibited, so that the finished product is essentially a park. It is not uncommon to find park systems in Eastern cities linked up with cemetery drives.

Scope of Planning Proposals.—The collection of survey material for a certain region will be influenced and guided by the aims and purposes of the designer or designers. Preliminary investigations will bring out the needs of the region and, to a certain degree, the extent of the planning proposals. The scope of planning proposals covering the general topics which are normally included in a regional planning program will guide the collector of survey data so that there will be less chance of gathering data "which will remain buried in the files".

The following outline was prepared for a county regional plan:

SCOPE OF PLANNING PROPOSAL.†

I.—Planning Services.—

(*a*) Preliminary Investigation:

A field investigation or local survey, by the planner and his associates, of the physical features, economic factors, social conditions, customs, and public opinion of the county, county seat, other towns and cities, and adjoining region.

(*b*) County Planning:

(1) Circulation: Lines of communication.

(*a*) Highways:

Comprehensive Highway System: Main highways (State and County); secondary roads; minor or local roads; special drives and boulevards; bridges.

Transportation: Buses; bus stations; trucking; trucking terminals.

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1507.

† Covering the general topics which are normally included in a regional planning program. The scope of the proposal, of course, would have to be modified to some extent to meet local conditions.

(*a*) Highways (*Continued*):

Suggested Road Improvements: Widenings, extensions; new roads.

Highways and Road Sections: Roadway widths; planting spaces.

Highway and Road Intersections: Study of important highway and road intersections.

Study of Special Problems in Relation to Highway Traffic.

Development of Highway Views.

Highway and Road Decoration: Lighting equipment; trees and shrubs; paving of roadways; removal of unsightly poles, wires, and signs.

(*b*) Railroads: The Place of the Railroads—Tracks, main lines, local lines, belt lines; stations—passenger, freight; grade crossings and grade eliminations; relation to present and future agricultural, horticultural, and industrial output.

(*c*) Water: Harbor and waterway terminals.

(*d*) Air Terminals: Selection of sites; requirements.

(2) Allocation of Areas for Appropriate Uses:

(*a*) Industry: Type of industrial development to best interest of the county; most appropriate areas.

(*b*) Areas for Colored Population: Present locations; probable future needs.

(*c*) Development of Water-Front (Ocean, River, Creeks, Ponds, or Lakes): Public and private use of shore land; residence; recreation; industry; commerce.

(*d*) Regulation of Land Sub-Division: Methods of securing proper development; acceptance of plats.

(3) Open Spaces:

(*a*) Parks and Parkways: Development of Parks and Parkways: Water-front parks; bird and game sanctuaries; neighborhood, or community parks; large parks, incorporating natural features as woodland of the region; connecting parkways and boulevards; promenades; park connections; small open spaces.

Linking up Parkways with City Systems.

Correlation of Open Spaces.

The Effect of Parks on Land Values.

(4) Schools:

(*a*) Standard Requirements for School Sites: Grade schools; junior high schools; senior high schools; agricultural schools; special schools and colleges.

(5) Recreation:

(*a*) Requirements of Permanent Population: Possibilities in Recreation: Playgrounds; county golf courses; athletic fields; swimming pools; beaches; piers; sailing and motor boating.

(6) County Building Sites:

Location of Buildings Important to the County.

Advantages of Grouping of County Buildings.

(6) County Building Sites (*Continued*):

Possible Sites for Administration and Resort Buildings, such as county court house; county auditorium; agricultural hall; county library; branch libraries; county fair grounds; county agricultural experimental stations; county institutions, alms, etc.; county fish hatcheries; aquariums; county jail; county dock; county railroad shipping centers; citrus exchange; municipal cultural centers for farmers.

II.—Zoning Services within the Area of Jurisdiction.—

(*a*) Preliminary Investigation:

Field investigation by the planner and his associates of the present use of land, density of population, height of structures, and public opinion of the county.

(*b*) Zone Plan:

Use Districts: Business; industry; residential built areas; agriculture.
Height Districts.
Area Districts.

(*c*) Zone Ordinance.

III.—Report.—

Typewritten report, with illustrations, setting forth the recommendations and supplementing the studies and plans. Overseeing printing of report for distribution.

IV.—Schedule of Plans.—

(*a*) Existing Conditions Map:

Showing the following: Highways; roads; parks; parkways; schools; playgrounds; public property; railroad facilities; industrial areas.
(A record of existing conditions at commencement of work and basic map for planning studies.)

(*b*) General Planning Study:

Outlines in a broad way the planning proposals.

(*c*) Preliminary Zone Study:

Preliminary districting for various uses of land, height of buildings, and area to be built upon.

(*d*) Study of Major Roads and Highways.

(*e*) Highway and Road Sections:

Suggesting appropriate widths of roadways, walks, and planting strips for the various types of streets.

(*f*) Zone Plan:

Showing proposals for use of land.

(*g*) Major Roads and Highways Plan.

(*h*) Parks and Recreation Areas Plan:

Showing all parks, parkways, boulevards, playgrounds, and park connections.

(*i*) Comprehensive Regional Plan:

Master plan of all proposals. Guide to all future developments.

(*j*) Sketches of Details:

Selected sketches necessary to supplement the general plans.

JOSEPH W. SHIRLEY,* ESQ.—It is evident that the author has given the subject of the requirements for a regional plan much real study, and has completely covered all the essential points. This is highly desirable.

Most cities are not so fortunate as the City of New York in having an organization such as the Russell Sage Foundation, which has been planned to conduct this and other similar investigations. In the majority of cities one great difficulty is to arouse public interest in a study of this kind and to secure funds, either by private subscription or from the public treasury, with which to conduct the investigations.

In the preparation of such a report, great care should be exercised that it be made in a practical, common-sense way, readily understandable to the layman who is not familiar with technicalities. It should not be filled with scientific matter. Furthermore, when the whole problem is presented, its immensity should not be such as to stagger the public and make it hesitate to carry out the recommendations. Another feature not to be overlooked is the item of time. Every one will admit the danger of continuing investigations over such a long period that when the time comes to act much of the information collected has already become out of date.

The data needed, as specified by Mr. Lewis, are highly desirable, but it does seem that each community must modify this information to serve its own needs, bearing in mind not only the appropriations required for the preparation of the report, but also whether the community can stand the expense in carrying out some of the ideals that engineers believe cities should have. After all, is not the public concerned with what is actually to be accomplished as a result of these exhaustive reports? So many cities have been "fed up" with elaborate reports on city planning, traffic, and other features that the inevitable outcome has been to shelve them after large amounts of money have been paid in their preparation. To put it plainly, it means to fit the plan to the pocketbook of the city.

It has only been within comparatively few years that the American public has been interested in city planning. Regional planning is justified by the results which have been obtained, even in these very few years, by proper city planning. However, almost as much missionary work will be needed to convince the community that it should not confine its plans within its present limits, as was necessary to interest it in a study of the area for its own immediate use.

For the collection of general basic data, it is fortunate for the people of this country that such an organization as the Russell Sage Foundation has been established. Much of the information gained by its exhaustive studies, a single community could not afford to collect for itself. The information concerning:

"(*a*) Highway capacities under different conditions
"(*b*) Recreation space requirement standards

* Engr. of Plans and Surveys, Dept. of Public Works, Baltimore, Md.

"(*c*) Length of haul of motor vehicles and transit lines
"(*d*) Standards of population and housing density
"(*e*) Variation in riding habit on transit lines
"(*f*) Model plans for a neighborhood unit"

outlined by Mr. Lewis,* could be accepted by every municipality making a regional study if this information is available to those who desire it.

One of the essential things which should be done by any city contemplating city or regional planning is to supply itself with a thoroughly reliable and complete topographic map. If the regional area is large, a map on the scale of 1 mile to the inch would serve very well. If that area is small, a larger scale map would be more desirable. The making of the proper kind of topographic map is expensive, but this is an expense which cannot be avoided. If there are no U. S. Geological Survey maps, or State maps, available the general map of the smaller scale can often be obtained at a relatively low cost from an aerial survey. These aerial maps can be enlarged and topography added in various neighborhoods as needed. In studies in Baltimore, Md., it has been found, where detail is necessary, that a topographic map on the scale of 200 ft. to the inch works out very well. This scale is small enough to place the situation on a workable size sheet at the same time covering a large area. From this scale map, work can be carefully planned and many calculations made.

For information concerning population and future distribution of population the zoning maps of the city are very useful. In some cities, public service companies have compiled much valuable data. Many railroads have perfected plans for their future development which are kept under cover for obvious reasons. An effort should be made to secure the confidence of these companies by those conducting the investigation so as to gain this valuable information.

Let engineers bear in mind that results are wanted and that care must be exercised not to spend too much time in making preliminary studies and elaborate reports.

GUY WILFRID HAYLER,† ASSOC. M. AM. SOC. C. E. (by letter).‡—This interesting paper opens up a wide vista of the scope and methods necessary for the successful completion of regional planning work. Regional planning is, as Mr. Lewis states, "a complicated piece of work", and the methods of one locality can not, and should not, be necessarily applied to another locality. Every city and every region is entitled to a personality of its own, and it would be deplorable to standardize cities so that they would be more dull and uninviting than they frequently are. While regional planning may be said to be physical in its groundwork and economic in its approach, there is also an underlying factor to be taken into consideration, that is, the recognition of the "soul of the city". This alone can give any plan "the punch" necessary to make it sufficiently appealing to the taxpayers who have to pay the bill.

* *Proceedings,* Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1507.

† Planning Engr., Regional Plan Assoc., Inc., of San Francisco Bay Counties, San Francisco, Calif.

‡ Received by the Secretary, October 16, 1926.

The basic information needed for regional planning is necessarily limited by the amount of money that can be expended on research and the personnel of those engaged in the studies. Then, again, the treatment of the subject by an unofficial organization will be different from that of an official authority. From the point of view of research it might be doubted if an unofficial body could do as well as an official body, but there would most certainly be an advantage offered by the former in that it could explore into many unknown fields, initiate new methods, and display greater originality. From this point of view the methods of the New York Regional Plan seem likely to establish many definite features which any regional plans of the future will be obliged to consider. It is very fortunate that the New York plan is being undertaken by such an important body as the Russell Sage Foundation which is National in scope and has considerable funds to prosecute its work. This should stand out as a significant fact to other localities proposing to do regional planning work and proclaim to them that a large sum of money is really necessary and the most highly trained experts are required, if studies and recommendations for regional development worthy of the name, are to result. The greatest failure of city planning in America has been due to the large number of communities that have started on the work with insufficient funds and negligible personnel, resulting in apathy in a very short time and no appreciable record of progress. Unless energetic city planning bodies are operating throughout a region, an all-inclusive planning body for that region is faced with a herculean task. If cities do meet the regional planning body half way, it is surely possible to eliminate many municipal problems and devote the time and money to the better purpose of establishing the wider co-ordination. It is necessary to think of regional planning from this angle if any foundation is to be established for the basic information. Of course, it is assumed that such regional planning is proposed in regions that are fairly well settled and comprise at least one large town or city as a center and scattered towns or suburbs in its sphere of influence. Assuming then, that a reasonable amount of money can be guaranteed over a period of years, and that a number of qualified men are available to undertake research and planning work, the general basic principles can be formulated.

Regional Plan Association of San Francisco Bay Counties has been in existence since 1924 and operates in a district composed of towns and cities of a strong individual character with a surprisingly small civic outlook as the city planner sees it. It has, therefore, been a matter largely of pioneering, where the education of the people comes first, and with it, the grouping of those anxious to undertake the formulation of a wide comprehensive plan. Under these circumstances progress is necessarily slow, and internal city problems which should be outside the general scope of regional planning are first in consideration, owing to the fact that there are no local organizations offering the technical help required. This may be an inevitable situation and it requires some ingenuity to prevent the wider vision from being obscured.

Coming to the matter of finance which is a limitation to research and anything else, the San Francisco Bay Regional Plan Association has been fortu-

nate in securing the active co-operation of Mr. Fred Dohrmann, Jr., a well-known Pacific Coast merchant, who has undertaken to carry the financial burden until the work has been established. This period is now drawing to a close as the organization is establishing itself in the nine counties surrounding the Bay. This splendid piece of civic patriotism has enabled the technical groundwork to be laid, together with a commencement on base maps, survey reports, and the promotion of several features of a regional plan.

The work has been built up by establishing the sectional elements of the research required. These are: (1) statistical; (2) streets and highways and traffic; (3) transportation; (4) harbor and water-front; (5) industries; (6) housing; (7) public utilities; (8) public health and safety; (9) education; (10) parks and recreation; and (11) agricultural resources.

While in the main this is understood to be a physical survey, the economic and social sides will be covered in the detailed "working out" of the elements of each particular section, together with the legal and financial aspects. A regional plan may be said to be in essence the ideal plan, and it must be practicable, legally and financially as well as physically, to complete it satisfactorily. Laws, however, can be changed and will be changed if the authorities and the people see that it is to their advantage to change them and, in that sense, the legal side may be said to be of an ephemeral character. Finance, as it is involved in the cost of public improvements and methods of assessment, is also largely a matter of legal enactment once the reasonableness of the cost of improvements is recognized.

In the San Francisco Bay District the sub-divisions of research before mentioned are probably the minimum in extent and yet cover the area of regional development with the least waste of time and money in collecting, verifying, and putting into form the data and suggested improvements. There is no object gained in spending valuable time and money over facts and statistics that are not necessary to prove points at issue, because such facts and statistics are generally obtainable from their original sources by any one who may wish to use them. The writer considers charts embracing the main points of research to be studied as one of the most useful elements in developing planning data and would advocate the wide use of colored pencils on maps used in the field. In the hands of qualified men this will insure a certain standardized method of gathering the data. When the latter reaches the central office, it can then be readily transferred to base maps, office files, and special scrapbooks covering the various subjects.

The regional planner will probably be faced with the possibility of altering many existing maps so as to secure a uniform scale and he is then concerned with the best size of map to use. It is obviously impossible to lay down any particular size for this purpose, but the San Francisco Association has found it convenient to use a map measuring 4 ft. 3 in. by 3 ft. 4 in., covering the area within a 50-mile radius and drawn to a scale of 2 miles to 1 in. All smaller maps will bear some relationship in size to these foregoing maps. The technical details of map preparation have been so thoroughly devised by the organization responsible for the regional plan of New York that very little

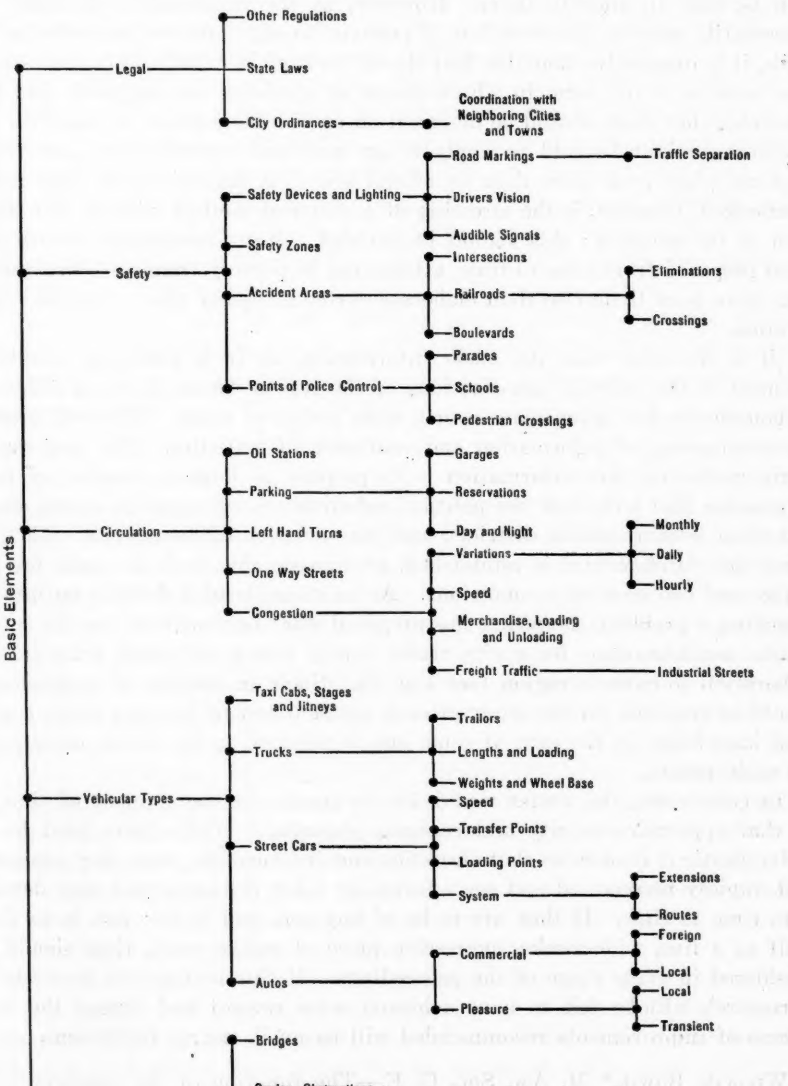

FIG. 3.—OUTLINE OF A CITY TRAFFIC SURVEY.

can be said to amplify them. However, as the information obtained, will necessarily receive the attention of persons largely unversed in technical details, it is imperative that this fact should be kept in mind. This qualification also applies to the form in which charts of statistics are prepared, and it is desirable that these should be designed on as graphic a basis as possible. The foregoing might be said to apply to an unofficial organization preparing a regional plan, even more than an official body. A danger which must not be overlooked, however, is the stressing of a pictorial method without due attention to its accuracy; this should be avoided. Many misleading charts have been prepared from time to time, attempting to portray trends of development that have been little else than elaborate cartoons, rather than scientific explanations.

It is desirable that the basic information, as it is gathered, should be grouped in the political sub-divisions of the region where the least difference in boundaries has taken place over a wide period of years. This will prevent the overlapping of information and confusion of statistics. The best way to begin gathering this information is to prepare as large a number of maps as possible that will show the political sub-divisions of counties, towns, cities, and other administrative districts, and the variation of boundaries with time. Once this fundamental is established, an unassailable basis is made for statistics and contemporary conditions. As an example of a definite outline for attacking a problem, the writer has prepared what he considers are the fundamental considerations for a city traffic survey which will need little further elaboration to cover a region (see Fig. 3). Such an outline of requirements should be prepared for the study of each problem and in drawing them, a wide local knowledge on the part of some one is required to lay down, tentatively, the main points.

In conclusion, the writer would like to emphasize the element of time in all that appertains to city and regional planning. While there need be no undue haste, it is obvious that statistics and information, once they are gathered, rapidly become old and are affected by other elements that may develop from time to time. If they are to be of any use, and if the plan is to show itself as a live, wide-awake, aggressive piece of public work, time should be considered in every stage of the proceedings. If this is done the heavy labor of research will be felt to have achieved some reward and indeed the final success of improvements recommended will be much nearer fulfillment.

William Bowie,* M. Am. Soc. C. E.—The function of the surveyor is to make his computed positions, his maps, and his plans useful to other engineers. His province is to supply exact knowledge of the terrain, in place of approximations and guesswork.

They are facing a heavy responsibility who are studying the physical needs of modern cities and who are directing the enunciation of the principles which bear directly on a city's growth and on the activities of its citizens. Exact knowledge is no handicap to a city planner, even when he is dealing with such general questions as street and park layouts and suburban communications.

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The laws of urban growth and development are imperfectly understood, and it is possible that a close study of detailed topographic conditions will reveal influences to which much consideration must be given.

The fiscal ally of the city planner, the tax assessor, can be easily persuaded of the value of an accurate cadastral survey which will show, in convenient form, all taxable real estate and the improvements thereon. Many instances are known where the increase in the taxable real property, disclosed by an accurate and complete city survey, has paid for the survey and left a considerable margin. The city planner and the city engineer need not show, in dollars and cents, the value of an accurate survey to their own particular activities.

The problem then, for the surveyor, is to furnish a survey with its resultant maps, plans, elevations, and positions which will serve the purposes of the city planner, the city engineer, the tax assessor, and all the general engineering interests of the modern city.

HOWARD STRONG,* ESQ.—In the discussion of this paper, two extremes, perhaps, were presented—one represented in a degree by the New York position; the other possibly by the Chicago method. The one would subordinate or even entirely disregard the various specific projects that may be before the region and would devote its entire attention to the studies preliminary to the making of a definite plan. The other seems to subordinate the surveys and studies in preparation for the plan and to emphasize the promotion of specific projects. There is a possible danger in either one of these extremes, even if, as in the case of the New York Regional Plan, finances are ample to go ahead with the making of studies and plans without regard to the attitude of the region.

If too much time is given to the making of the plan, and the actual planning accomplishment is entirely neglected for a number of years, it seems likely that the interest, enthusiasm, and support of the people in the region, upon whom, after all, depends the adoption of the plan, is likely to be lost. On the other hand, if too much emphasis is put on the project method, the picture is likely to get out of perspective, and the projects, themselves, will not be properly articulated in the general scheme.

It is certainly necessary to retain public support and, for that reason, some attention must be given to programs in which the people have an immediate interest. At the same time, the plan itself, which, after all, is the principal business in hand, must not be neglected. The only solution that will develop an adequate plan and will, at the same time, retain the interest of the people who must ultimately accept the plan, is a nice balance between the two extremes.

* Secy. Director, Regional Planning Federation, Philadelphia, Pa.

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PAPERS AND DISCUSSIONS

HOUSING AND THE REGIONAL PLAN

Discussion*

By Messrs. James Ford, Benjamin H. Ritter, Bernard J. Newman, Bleecker Marquette, W. C. Rice, E. M. Bassett, Frank B. Williams, and U. S. Grant, 3d.

James Ford,† Esq. (by letter).‡—This paper is the ablest discussion of the civic and social aspects of housing in its relation to the regional plan that has come to the writer's attention. It shows clearly the paramount importance of making the regional plan contribute to human welfare and civic progress. The author outlines specific ways of escape from the present deplorable tendency to let selfish interest dominate public interest.

In America, ugliness and unsightliness in industrial and residential districts, and noise and inconvenience in traffic arrangements, have become so common that they are taken as matters of course. There is no reason why access to the amenities of life should be the exclusive right of the well-to-do. If the highly trained minds of the Engineering Profession can be applied with equal force and skill and practical judgment to the problem of providing better homes and better neighborhoods and better community life for wage earners, America will make over its industrial districts so radically that they will be a source of justifiable pride.

Mr. Ihlder points out effectively that the next generation should be the object of chief concern. Progress and America's contribution to world civilization rest with them. They are largely the creatures of their environment and cannot far transcend it. The drab, monotonous, unkempt, crowded, treeless, industrial quarters of American cities can hardly produce enthusiasm for public service or good judgment as to what constitutes civic interest. Housing conditions that make for wholesomeness in home life and respect

* This discussion (of the paper by John Ihlder, Esq., presented at the meeting of the City Planning Division, Philadelphia, Pa., October 7, 1926, and published in September, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Executive Director, Better Homes in America, Washington, D. C.

‡ Received by the Secretary, October 16, 1926.

for government are, therefore, necessary and the regional plan is clearly the first point of attack.

Sunless, unventilated, crowded homes produce irritability and low vitality, if not actual ill health. The detached dwelling, placed with careful consideration of orientation, makes it possible to have sunlight and cross ventilation in each room and provides for adequate privacy and control of the factors of home life. It also makes possible the home garden and the safe home playground. It can provide relative freedom from the dust, noise, and accident danger, which characterize crowded residential districts. It unquestionably provides larger opportunities for self-development on the part of parents and children. The home ownership, which it makes possible and which, for the majority of the population, is advisable, contributes to civic interest, for the home owner ceases to be a nomad, puts down roots in his neighborhood, recognizes community and civic interests, and bends his energies to the improvement of his neighborhood and of his city.

Carefully considered regional planning, as outlined by Mr. Ihlder, will, in time, make these values accessible to all. There are few civic measures more significant than this.

Benjamin H. Ritter,* Esq.—The author has outlined the basic principles of regional planning, which must be observed in any metropolitan district if housing is to be developed and maintained on a sound basis.

Surely all will agree that adequate open spaces surrounding accessible individual homes with sanitary equipment and modern conveniences, are the primary requisites of good housing. As indicated, this requires the proper distribution of population over a given area. Therefore, the outstanding need of any region, from the housing standpoint, is the adoption of a practical plan that will facilitate the conveniences of the home and prevent congestion and overcrowding of the land.

In the absence of plans, regulations, and possibly some of the necessary equipment in the past, many centers of population have been allowed to become overcrowded to the extent of 100 to 700 persons per acre. These conditions are not limited to Manhattan Island, or to any large city. They are found even in the small mining towns of Pennsylvania. Such practices have resulted in what has been termed, very fittingly, "the American blight." Unquestionably, the time has come when the authority vested in municipal governments must be exercised to check and control this injurious form of city growth.

The problem does not seem so complex when it is realized that in the United States there are 50 metropolitan centers, each with a population of 150 000 and more, scattered over 3 000 000 miles of land, most of which is habitable. These centers with their surrounding territories represent the country's metropolitan reservoirs. Knowing where the reservoirs of people are and the topography of the land surrounding them, it should be more or less easy to determine the flow of population and properly to provide for it.

* Executive Secy., Pennsylvania Housing and Town Planning Assoc., Philadelphia, Pa.

The machinery for handling this job is in the making. No one ventures to say that it is perfect, but the various experiments made with it, and the many new parts patented during the past ten years, convince one that it will be effective. Its successful operation will depend on the willingness of individual urban communities to co-operate as a unit under the guidance of some form of a regional organization.

To make it effective and carry the proper weight, it seems that membership in this regional organization should be by election or administrative appointment from the various municipalities concerned. A nominal fee for attending meetings might not be out of order. This board with its complete data and maps, showing the general layout of the region, elevation of territory, incorporated villages, transportation lines, present use of land, the location of industry, commerce, and residential districts, open spaces, population density, and curves of population growth, will be in a position to direct the proper development of the region, preserve the individuality of the small towns, and yet enable them to carry on their various public activities as one large unit.

A properly laid out street and road system will serve as a reliable guide for future sub-divisions and platting of land. Parks, playgrounds, club houses, shops, stores, and all the complex equipment of a modern city can be located with a definite economical relation to each community center as well as to the district as a whole. This will encourage regional light, power, gas, water, and sewage systems, in addition to many other community enterprises, both public and private. It will tend to reduce the cost of supplying necessary conveniences to suburban districts. With the coming of the automobile, motor-bus line, and adequate suburban streets and transportation lines, isolation will give way to united communities with all the advantages found in any city plus a more delightful home environment. This will give rise to incentives that bring about the proper decentralization of people and will make it economically possible for families to live in homes of better selection.

Even now, the doors of the suburban districts are opening to thousands of middle class families even if the bread-winners must continue to work in the city. It remains the duty of each incorporated unit in metropolitan regions to safeguard this movement by protecting all residential districts with proper zoning regulations. Likewise, it is the obligation of government to see that sufficient space for industries is reserved in some of the resident districts, so that employment may be provided for those who may wish to work near home. Here, again, a guiding hand for the metropolitan region is of greatest importance, for it may be just as necessary to advise a municipal unit what not to do, as it is to advise it what to do.

It must be realized that industrial plants and even a variation of housing accommodations are just as necessary as individual homes. One cannot survive without the other. A properly regulated factory or an office building may be just as essential to one locality as a hotel or an apartment house is to another. The complexity of modern life will not permit any of these factors to be overlooked. The manner in which they are correlated will determine the future housing in metropolitan regions.

BERNARD J. NEWMAN,* ESQ.—The author, looking out over an expanse of slum waste in American cities presents a vision by which, if consistently held, home life may be changed so that it will produce the man power needed to maintain the supremacy of the nation. Large areas in American cities are a waste to-day. There is no justification for herding tens of thousands of families in congested centers. The inhumanity of it is apparent. The social loss is measurable. In Philadelphia, Pa., for example, more than $28 000 000 are spent annually to care for defectives, dependents, and delinquents. That is not a foundation, but an annual expenditure of public and private charity for the care of the waste product of urban living. Under the right program, of which planning is a part, much of it can be salvaged for capital expenditures.

The paper assumes that the fundamental note of all city or regional planning is human welfare. Through highways are recommended; parks, parkways, and recreational centers are suggested; the unification of sewer systems is proposed; and common sources for regional water supplies are advocated, not as technical engineering problems, but as adjuncts to serve man in the various aspects of living.

The author has implied that the regional planner is more than a technician with an engineering problem before him. His technique, based on the fundamental principles of his profession, is essential to the solution of his problem, but it is secondary to its object, namely, the layout of the area for constructive communal service. If this is accepted, the logical deduction is that any regional plan proposed is defective if it is not evolved from a concept wherein the human side of communal life is of more concern than the mechanical aids toward its expression. Such mechanical aids are not thereby minimized in importance, but are properly subordinated to their place as aids. In other words, the physical city as it is to-day is of no value except as it is a place in which man may live, work, or play. The hollow echoes of abandoned mining towns stir only historic interest. Their street plans, business areas, and housing are of merely academic significance. Man has forsaken them. Cities are concerned with physical expression, solely because of suitability or non-suitability to meet the every-day needs of those who there live, work, or play. Manifestly, in their planning or re-planning, the object of prime importance is that they may serve man in each of his major interests with the least restraint or injury. If they do not so serve him, then the logical and the practical thing to do is to control all future urban development or re-development so as to avoid past mistakes.

In determining the significance of all that comprises an urban area and in planning for its utilitarian as well as its artistic expression, the fact must not be lost sight of, that the preservation of family life, which the sociologist declares is the unit of all social life, is of fundamental and primary importance. In short, highways, transportation facilities, parks and civic centers, are not entities in themselves, but are correlated units, obtaining their real value from the service they render to their community. To the extent that

* Managing Director, Philadelphia Housing Assoc., Philadelphia, Pa.

their construction or placement is unsuitable, or produces a problem in any other phase of communal life, they are subject to remedial action; to the extent that their planning or promotion fails to take into account this interrelationship, they are faulty. There is an interdependence between the physical expression of the city, in its provisions for industrial and commercial needs, and those other aspects of such physical expression manifest in residential areas and the social needs accessory thereto.

This fact has not received adequate consideration in city growth. If it had, there would be little need for legally expressed regulation for zoning; multiple housing would not have gained headway; there would be no such widespread areas of slums; cities would not be so concerned with population "bigness"; and there would not be such concentration of land values with all the attendant increases in the cost of production and government. In like manner there would not be so many sectional plans concerned with getting a business population into or out of a business center. Things as they are would not be considered fixed, with attention given to alleviatory plans which, as in the subway in New York, or the loop hazard in Chicago, often serve to accentuate the problem.

This thought must be kept in the foreground of all proposals for city development advanced by the city and regional planner. Has it been so kept? The speaker does not think so. It has been subconsciously accepted, but there is little evidence available in proposed plans to justify the belief that it has been an ever-conscious companion of the planner's thoughts. For this reason, therefore, it is very pertinent to analyze the place of housing in the regional plan.

Mr. Ihlder has very properly pointed out the illogical development of cities to-day with land loads, in building and people, that have placed heavy burdens on family life, with a slum virus that has not only menaced communal health, but destroyed the industrial efficiency and commercial resources of hundreds of thousands of workers and their families. He has challenged the practicality of congested business districts and concentrated industrial areas. He has emphasized the necessity for decentralization, pointed out its feasibility, and outlined means whereby the present trend may be expedited through satellite communities, interrelated with, but separated, geographically and physically, from the parent city. As a natural corollary, he has indicated the compelling necessity for suitably planned and adequately supplied housing accommodations.

Cities are not fixed entities, but units, breaking down in spots by natural processes through the withdrawal of industries to more suitable sites and the migratory practices encouraged by business concerns chasing each other to more strategic centers of trade. A constant process of wedging in and drawing out is written in the records of the decadence, alteration, or demolition of buildings in all older areas. A financial district may remain static in a city like New York and the insurance district change, or the insurance district may have sent its roots down and the financial district changed, as in Philadelphia, but there is constantly going on, in all American cities, that illusive maneuvering for the "best" locations, which is conclusive evidence of faulty

planning. The greatest loss is to the home life of the people, which is ruthlessly encroached upon. The dwelling, in which such home life is centered, loses some of its desirability. Its occupants, in order to preserve those attributes of home life most prized, are forced to move to other areas farther away from centers in which they trade or labor. Regional planning, therefore, offers the opportunity to reverse this mistaken policy, especially in new areas. In the almost virgin territory outside large cities it can plan for those satellite communities which are practically complete entities in themselves, but are so connected with the parent city by highways and transit facilities that the ultra-cultural advantages offered by the latter may be available to the former. In such satellite cities, interrelated and co-operating with each other for the economic operation of service needs, will be found the antidote to the lure that over-populates urban centers while depopulating rural areas. There, adequate housing can be established and protected without loss to or encroachment on industrial or commercial life.

Mr. Ihlder has outlined the essential elements, industrial and commercial, cultural and recreational, that are needed in such smaller communities, and has emphasized the feasibility of providing satisfactory home environment with space in and about the dwelling to serve family needs. Decentralization is the keynote. Its popularity cannot be questioned in the face of the present higher rate of growth of suburban towns as compared with the cities to which they are adjuncts. The trend is that way, but it needs wise directing in order to obtain for the future citizen the best results. The practical difficulty, under present land-development practices, of maintaining the necessary isolation from the parent city is remedial. The suggestion is made that intercommunity parks separate geographical boundaries and that, by low land cost, agricultural belts be established which, because dedicated to agricultural use, cannot rise to the prohibitory land values accompanying other forms of development. The speaker doubts whether there can be any insurmountable legal disqualification to the establishment of such community safeguards. The processes involved to secure them are: By voluntary gift of civic-minded property owners, or landlords and others interested, of land for park purposes, specially designated for all time for such uses; by municipal purchase, especially of areas where land costs are lower; by excess condemnation when boulevards or transit lines tap undeveloped districts; by zoning; by prior designation of use, as practiced in small park and street platting in Philadelphia, prior to the actual taking title thereto by the municipality, so as to forestall the erection of private structures with an attending higher condemnation cost to the local government; by preferential tax rates and assessments of land according to city, suburban, or farm use; and similar methods which have been adopted where public control over land has been exercised by towns and cities.

The author has discussed many phases of the part that housing plays in the regional plan. The major contribution he has made is his emphasis on the need to expedite a movement already under way and to guide it so as not to create a flock of evils in new centers similar to those so costly in the old. He clearly sees that the promotion of housing betterment in urban centers

must always labor under the handicaps of faulty planning, or lack of planning, of the past and although relief can be secured, nevertheless compact construction dooms family life to be satisfied with less than the normal advantages essential to wholesome living.

BLEECKER MARQUETTE,* ESQ. (by letter).†—This scholarly paper goes to the root of the housing problem. There can be no question in the mind of any one who has had experience in this line, that there are no arbitrary city boundaries that can confine within their limits the housing problem of a city.

The importance of the proper allocation of industry and homes for workers is quite properly stressed. It is desirable to eliminate, at least to some extent, the severe strain on the vitality of workmen in the long period of unnecessary travel to and from work. Few people have any real conception of the ill effect of the terrific hustle and bustle of modern city life. The fact that mental disease is found to be uniformly more common in cities than in rural areas is enough to justify careful study of ways and means of decreasing the strain of modern city life.

The writer is not discouraged with the possibilities of regional planning but it will take a great deal of effort and much education before the attainment of real progress toward Mr. Ihlder's ideal of locating industry and housing so that the amount of unnecessary time spent in travel will be cut to the minimum. Perhaps, some day leaders of industries will see that they have much to lose by locating in large cities, and will seek smaller communities where their workmen will have an opportunity to enjoy the amenities of living. At the moment it must be admitted the trend seems altogether in the opposite direction.

Mr. Ihlder presents a unique point of view when he discusses the difficulty of finding logical use for land adjoining main highways in cities. This is a great problem and will become a greater one. Certainly, business will never occupy all the properties along these highways. Zoning laws usually locate apartment districts adjoining them. Tenants of such apartment buildings are already beginning to recognize the disadvantages of the noise, dust, and commotion, and architects and builders are reflecting their objections. They are demanding that sites be provided for apartment houses away from main highways. The suggestion, made by Mr. Ihlder, that minor streets be laid out parallel with thoroughfares, separated from them by parked strips, and that apartment houses be constructed along these minor streets seems worthy of consideration.

The housing of a city cannot be safeguarded without regional planning. That was learned early in the game, before anything was known about regional planning. Builders soon find that limitations are placed on the shack type of house in the larger cities, but that they can step outside the city limits and build just about as they please. The better the housing regulations in the city, the more temptation for "jerry builders" to carry on "beyond the pale". Nothing is more distressing than this to the person active in housing

* Executive Secy., Cincinnati Better Housing League, Cincinnati, Ohio.

† Received by the Secretary, October 16, 1926.

improvement in a community. Just as he has begun to bring about a reasonably satisfactory situation in the city, he finds himself confronted with a serious housing problem in the outlying district. This is exactly what has happened in Cincinnati, Ohio. The city has a reasonably good building code, and the housing regulations are now in the process of complete revision. It has perhaps as intelligent and conscientious enforcement of the regulations as could be asked for by any city. It has a good zoning law and a comprehensive city plan. In the meantime, while attention has been directed toward problems within the confines of the city, little slums have been growing up in the county. Some method of controlling the development of these areas is becoming absolutely imperative, and the regional plan is undoubtedly the way to meet the problem.

Cincinnati has eight distinctive satellites, several of which are nearly surrounded by the city. They are practically a part of the city life, participating in its benefits and materially affecting its welfare. Most of their inhabitants earn their living in the city, use the city's streets, and profit by practically all the safeguards of city government. They do not want annexation. A regional plan would help to make it possible to fit these satellite communities into the whole and to bring about a situation in which they will play a proper part in the city's life and in the maintenance of its government. A State law is already in effect which provides for the establishment of a county and regional planning commission. Forward-looking citizens interested in the proper development of Cincinnati regard regional planning as the next big forward step.

W. C. Rice,* Esq.—As Mr. Ihlder points out each regional plan will have to meet the conditions of its local setting. The speaker desires to indicate some trends affecting the Pittsburgh, Pa., Industrial District.

General Features.—The Pittsburgh District owes its general layout to its topographical features, which determined its location on account of water transportation. The railroads follow the waterways, giving the District its present formation in which the lowlands have been taken over by industry and the highlands by residential districts. Future development will continue along these lines, with the unoccupied lowlands being taken up by industries. The railroad connections available are now generally supplemented by an improved system of highways. Therefore, the main river valleys and the valleys of the larger creeks, in which railroads now exist, will form the industrial sections of the future Pittsburgh Metropolitan District. The highlands between these valleys form the logical location for residences and the sections that lie closest to the valleys should be considered as the best location for the homes of the operatives of the adjoining industries. The more remote highlands should be preserved for the more expensive residential developments and for parks and other public recreational features.

Facts Influencing Location of Residences.—This is influenced, however, by the demand for housing accommodations of those who are not directly concerned with the operation of the industries, but who serve the industrial

* Pittsburgh, Pa.

population as a whole in the various commercial enterprises. It is also affected by those who, on account of an improvement in their economic condition and the diversity of occupations in the family group, are not located in close proximity to the mills where they work. The changing conditions in employment and the improvement in living conditions have been the great influences that have changed neighborhoods.

Proper Basis for Area Requirements.—The present practice of zoning on the basis of the family unit is not as logical as zoning on the basis of the individual. The classification suggested by Mr. Ihlder is not comprehensive or elastic enough to form a proper basis for a housing-control program.* There are certain minimum requirements of area and volume per individual that govern, and if a reasonable limitation is made as to area per individual, the multiple house will lose the greater part of its objections. Ample area and street frontage would then be available, and the light and air space of the adjoining property owners would not be injuriously affected.

In neighborhood commercial districts, where the ground floors are used for business and the upper floors for residences, in addition to the minimum area per capita requirement, the lot should be of sufficient area to allow twice that occupied by business. For land located in the lowlands and which is obviously a future industrial site, the minimum requirements per capita should be considerably increased for the reason that these locations are generally not as healthful because of the prevalence of fog, smoke, and dust.

With these basic considerations determined, the early sub-dividers should be given wide latitude in laying out their property. Further development of the property by houses would then be easily controlled by proper building code and fire protection restrictions.

Multiple versus Single Family Houses.—The single-family house may not be the ideal one for a large industrial population. It is more expensive than the multiple house. If the speculative builder is eliminated and the cost of building construction continues at its present level, the opportunity of the individual to secure a single-family house ample for his needs will decrease. Then, if the accommodations are not furnished by the industries, doubling up of families and greater congestion than that which now exists, are bound to occur under the pressure of economic necessity. On the other hand, if the speculative builder is encouraged and controlled by proper building regulations and zoning restrictions, he will use proper materials in his buildings.

The housing of the Pittsburgh Regional District must depend more and more on the speculative investment builder and on the industries themselves. The growth of financial knowledge, the increasing mobility of people, and the changing ideal in the family will cause them to view with concern any tendency toward establishing themselves by a considerable investment in one definite place. This would tend to restrict their ability to take advantage of opportunities in other parts of the District or other sections of the country.

Some of the tendencies along this line, which must be recognized, are the greater investments being made in automobiles, radios, and similar facilities

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1521.

for enjoying life. Stocks, bonds, and similar investments, which do not require so much oversight on the part of the owner, are also gaining in popular favor. The prosperity enjoyed by the inhabitants of the industrial sections of this country—increased pay, shorter hours, restriction of emigration, and aversion to manual labor by the younger generation—are also dominant factors.

E. M. BASSETT,* ESQ.—City plan maps must be precise because they mark the different characters of parcels of land. The boundary of a park must be definite—a definite line; the boundary of a street must be a definite line; the boundary of a zoning district must be a definite line; so with the school site, a fire-house site, the site of a court house; and, also, with pierhead and bulkhead lines. It is the determination of those lines that constitutes city planning.

Open spaces between communities—how shall they be obtained? Mr. Ihlder has properly stated† that they cannot be parks, because parks are an expense to a community and yield nothing by taxation. These open development spaces must be put to use, as flying fields, fair grounds, race tracks, country estates, golf courses, etc. They probably cannot be obtained through adjustment of taxation. A low tax rate will cause increased value, because the land is favored by taxation. With any low taxation there must be regulation of the percentage of land to be covered by buildings. Probably a 3% limit of cover cannot be attained through the police power because the Courts would decide that it was a "taking" and not a police power regulation. Perhaps these open development spaces can be taken by the State imposing an easement that will prevent structures occupying more than 3% of the lot. Such an easement could be taken by eminent domain. Owners of country estates dread the incoming of developments of 30 by 50-ft. lots. Some of these estate owners would not make claims for damages if the State imposed an easement of 3% cover on the land.

This country is the only one where the acts of legislative bodies can be set aside by Courts, and, therefore, citizens can only do by legislation those things that can be brought under the police power. If it is not possible to accomplish the result under that power, it must be accomplished by eminent domain, and eminent domain is expensive.

FRANK B. WILLIAMS,‡ ESQ.—This paper has inspired a quickened sense of the importance of the proper regulation of the region as a whole and has strengthened the belief that by itself regional planning, in the sense usually attached to that term, is insufficient for that regulation.

There are many reasons why regional control is becoming rapidly and increasingly necessary.

First.—With the growth of cities, urban population is more and more seeking the remoter suburbs. Population is thus increasing not only in the centers, but in the entire region.

* Counsel, Zoning Committee, New York, N. Y.

† *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1515.

‡ Attorney-at-Law, New York, N. Y.

Second.—Throughout the region, and not in cities alone as formerly, people are demanding urban facilities, and local officials are finding the problem of satisfying these demands, and reconciling them with those of other localities, more and more difficult of solution. Urban problems, now region wide, are almost impossible of solution by local effort alone.

Third.—Urban facilities in rural and semi-rural localities are bringing with them poles, billboards, filling stations, "hot dog" stands, and other nuisances, prevalent in cities, to escape which city people have gone to the remoter suburbs. There is a deepening feeling that, to-day, these conveniences can be furnished in the country without giving the offense which is often regarded as necessarily incident to them. This, however, can be done only by restrictions covering broad areas.

Evidently the planning of a region or any other area is not enough. The plan, to be effective, must be enforced. The growing importance and difficulty of regional planning is bringing to the forefront, as never before, this problem of the administration of regional planning, which in this country still awaits solution. In so far as such administration has been attempted it has been of two kinds:

(1) The jurisdiction of the city has been extended a certain distance over the adjacent outlying territory. This has proved to be both too much and too little. It is undemocratic to govern one community by the vote of another, and this practice will not be continued. Indeed, in the more developed parts of the country where regional administration is especially important, it is not feasible. Thus, Nassau and Westchester Counties, New York, will not tolerate any control over them by New York City. Because it is undemocratic, it seems impossible, anywhere, to give the central city planning powers over the outside land, which are necessary for a sufficient measure of control. For instance, no central city is allowed to zone outside areas in spite of the great need of harmony in the zoning of the entire region.

(2) Regional control is attempted by extending the boundaries of the central city. This extension, often greatly delayed by local interests, is never wide enough to give the regional control necessary, and if it were sufficiently extensive for that purpose, would unduly destroy local government, so necessary for efficiency and happiness; for, with all its faults, home rule, in large measure, is more advantageous than centralized control of local affairs.

There seems to be no immediate prospect of the attainment of effective regional planning administration in this country. Abroad such administration exists, and is obtained by State supervision of local planning, harmonizing local conflicts, and furnishing the incentive to local governments in their work. It is along this line, it would seem, that real regional planning must come in America. To some it seems contrary to the principle of home rule; but, in fact, it preserves home rule by giving just the proper amount of regional control without further interference with local government. Other systems fail by reason of ineffectiveness or excess; State supervision succeeds by keeping to the golden mean. Successful abroad, it will surely come in this country.

U. S. Grant, 3d,* Assoc. M. Am. Soc. C. E.—The speaker, whose practical work with city planning has only covered a few years, has been engaged in some regional planning which includes the National Capital, and some rather peculiar problems that have arisen there.

In the first place, it appears that city planners are likely to be too intent upon the problems of the moment, especially traffic and business problems; and that the urgency of a solution to these problems is very apt to lead any planning commission from the more general, and to the speaker's mind, more important, problem of planning for the future. Work on the two must go on simultaneously, and one cannot be neglected in order to take care of the other. Moreover, the more immediate problems demand attention and, therefore, are sure to be solved in the natural course of events, so that the really serious problem, which the city planning profession alone can appreciate and handle, is that of preventing too great a concentration of population, and the evils that come therefrom; that is, the problem of preserving the open spaces. Parks alone, and the high cost of securing them, even by exercising the right of eminent domain, will hardly save the situation for the present; although it may take care of it in the future, if some other way can be found to increase the available amount of open spaces to meet present needs.

A partial lien on open spaces used for other purposes certainly does indicate a legal method of preserving all the open space that is still left. However, there is another legal method of carrying out the combined operation of different authorities, as in the State of California, where a plan of flood control, extending over nearly all the northern part of the Valley of California has been adopted jointly by the Federal Government and the State. Each has set up a board with authority to modify the approved plan to some extent, and each has passed laws by which plans inconsistent with the adopted project cannot be carried out. In the case of interference of this plan, with irrigation and other local projects, the proponents of the latter can, of course, have recourse to the Courts and show a damage which is covered by appropriation or assessment; but thus far this method of prevention has worked very well, and the Supreme Court of California has sustained the legality of the legislation. It offers a compromise measure which is immediately possible where planners have to deal, as they do in the region of which Washington is the center, with municipalities or States and the United States. Even in other cases, where separate States mutually agree upon some general plan for the preservation of certain open spaces, each contributing toward its cost an amount proportioned to the features in which each individual State is particularly interested. Each State would constitute its own board to carry out its part of the plan; and both boards, meeting together, could arrive at a mutual decision as to general changes in the plan, which were found in the interest of both sides. Within the limits of its own jurisdiction each State Board would then prevent any development inconsistent with the project during the period necessary to complete it, and increases in taxation of open

* Lt.-Col., Corps of Engrs., U. S. A.; Director, Public Bldgs. and Parks of the National Capital, Washington, D. C.

spaces remaining in private ownership could be prevented, wholly or in part, as long as their open character beneficial to the public is maintained.

The question of the cost of parks is one that deserves more study than it usually gets. Those who come into city planning from the Engineering Profession are especially inclined to look upon parks as a luxury that is expensive and, therefore, something that should be dispensed with as much as possible. In many cases, especially where they are laid out after careful study, parks may actually result in an economy to the city in which they are placed. One park project, over which urban development would have been very much more expensive than park treatment, has been before the City of Washington for about twelve years, and is now nearing completion. The only possible loss to the city because of the park is the reduction in taxation due to losing the assessments that might have been raised on the property located within the park limits; but no such loss will actually occur because the increase in taxable values of the land adjacent to the park may be expected to equal the former value of the land taken, not to mention the benefits in traffic and other facilities which it offers. There is an actual gain of about $4 000 000 to the city in the cost of the park as compared with what it would have cost to make the usual street developments.

The open spaces must include, not merely playgrounds and small city square parks, but larger areas. The necessity for them is very much greater than it was, because the public has a much greater amount of leisure time to spend out of doors, which has to be provided for. Twenty-five years ago the laboring man worked 10 to 12 hours a day, and his children began to work quite early in life. Nowadays, those children are still going to school at the age of 14 and 15 during part of the year only, and then just a few hours of the day, and the working man himself has a considerable amount of leisure time during the day and in the evening. They cannot find a safe or satisfactory place for exercise on the streets, because of the crowded condition, and the parks are their only safe and sane recourse.

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PAPERS AND DISCUSSIONS

CULTURAL OPPORTUNITIES IN REGIONAL PLANNING

Discussion*

By Messrs. Henry V. Hubbard, James Sturgis Pray, Noulan Cauchon, E. M. Bassett, Jacob L. Crane, Jr., Charles Wellford Leavitt, Morris Knowles, Harold M. Lewis, R. H. Randall, and C. H. Howell.

Henry V. Hubbard,† Esq.—The author emphasizes the idea of bringing the parks to the people. Every one agrees, but the process has its limits. Books can be brought to people because the book is the same whether one reads it in a great public library, in the branch library, or at home; but the essence of a big park is its bigness, and that cannot be brought to the people by distributing the same recreation area among their homes in small fragments. The playground and the little park can be brought to the people. The people must be taken to the big park.

Mr. Crawford speaks very truly of the desirability of zoning the region with due regard to the park areas. This principle should be applied also to the uses permitted in parks and parkways after they have been acquired by the public; that is, subsidiary uses destructive of their main purpose should not be allowed to invade the park areas. Landscape charm, reasonable quiet, and safety are essential attributes in parks and parkways. The cultural value of the parkways to the motor-truck drivers, whose welfare Mr. Crawford so feelingly bespeaks, would not be sufficient to compensate for the destruction which the noisy dangerous passage of the trucks would work in those qualities of the parkway which fit it for its main and essential recreational use.

As to the engineer's contribution to city planning, it should be remembered that the engineers of the early Nineteenth Century were fighting alone in the front line of the battle. There were at that time no art juries, no city planners, and no lawyers trained to give counsel to engineers and to help

* This discussion (of the paper by Andrew Wright Crawford, Esq., presented at the meeting of the City Planning Division, Philadelphia, Pa., October 8, 1926, and published in September, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Editor, *City Planning Magazine* and of *Landscape Architecture*, Cambridge, Mass.

them in their difficulties; but even so they were not without their triumphs in the field of esthetics and of broad plan conception. Major L'Enfant, for instance, was an engineer.

As to the future of engineers in regional planning, it is plain that whoever lays out the broad conception of future community development, his work will be bad unless it is based on engineering knowledge; and, in any case, most of it will be built by engineers.

The main lesson of this discussion is obvious. For the fuller development of all the cultural opportunities of the city and the region, there should be fuller co-operation among all the professions involved; each profession knowing that it is only one wheel of the city-planning cart. The cart would not run well without that one wheel, but it would run still worse supported on one wheel alone. Many examples of effective common sense co-operation between professions in regional planning may be seen, and it will not be long before it will be a matter of course.

JAMES STURGIS PRAY,* ESQ. (by letter).†—This topic opens up an enormous field. Mr. Crawford brings out effectively and suggestively many of these cultural opportunities. Culture may mean very different things to different people—in short, anything from *Kultùr* to refinement of taste. If the development or strengthening of the mental powers is meant; or improving or refining of the mind, the morals, the taste;‡ and not merely the culture and refinement of the few, but rather the advancement and uplift of the many; then, surely comprehensive, competent, and far-seeing regional planning not only offers unlimited opportunities for culture, but is essential to the most widespread culture. It is known that culture, proverbially, has bulked larger in cities, but cities have no monopoly of it. Some of the most truly cultivated minds are possessed by individuals who have spent their whole lives in the country, and some by those who have lived a large part of their lives in the wilderness, for example, John Muir. The region, the term as now commonly used for a great metropolitan area with a city center and extensive outlying suburban and rural districts, embraces both city and country. The whole strong aim and tendency of regional planning to-day is, not only to extend into the country as many as possible of the advantages long accruing almost exclusively to city dwellers but, also, to bring into the city all possible of the advantages of life in the country. This is far more important. All the opportunities for culture offered by the physical and human environment may then be provided by regional planning, for it is not necessary to except wholly even such as may be had only in the wilderness, since areas in a wilderness can be conserved by planning for their use; as, say, in the region of Geneva, Switzerland.

Any region will, of course, even without what is called regional planning, offer many of these opportunities. That is, it will offer these opportunities in ways limited either to certain classes of the population or to all classes in

* Charles Eliot Professor of Landscape Architecture, and Chairman, School of Landscape Architecture, Harvard Univ., Cambridge, Mass.

† Received by the Secretary, March 18, 1927.

‡ Standard Dictionary.

certain respects only, and within certain none-too-broad limits. However, great metropolitan areas to-day, without regional planning, offer also much, and in some ways more and more, that works powerfully against culture. Crowding without planning, and the speeding up of life of which large populations tend to become victims, operate against health; the lack of planning adequately for ease of circulation leads to colossal inefficiency and waste; and the thoughtless piling up of ugliness in many parts of these great areas certainly dulls the popular sensitiveness to beauty and deadens even the aspirations for beauty in environment. Culture may exist in a slum or in a deadly monotonous suburb or in the most repulsive industrial area, but certainly these conditions in themselves do not make for culture; yet normal humans, the uneducated as well as the educated, the simpler minded as well as the more sophisticated, are responsive naturally to beauty in their environment and particularly to beauty in its universal and its highest forms. Given sufficiently widespread understanding and will to co-operate in bringing about a better set of conditions in the city or region, there is every reason why not only living areas and recreation areas to which much attention to beauty is already given, but all working areas as well, should supply a vastly more beautiful environment than they do at present.

The control by regional planning of these matters, when it includes regional zoning, will increase these opportunities for culture in the following principal ways. True culture comes both from hard work and from times of relaxation and is, perhaps, like genius, one-third inspiration and two-thirds perspiration. Sound regional planning will go far to promote and assure public health and the general physical well-being of the people living, working, and playing within a region. It will bring increased alertness, sensitiveness, and greater sanity, and will give to the workers at once more power to draw culture from their hours of labor and more energy to devote their spare time to things that make for culture over and beyond what their regular pursuits may bring them. Therefore, sound regional planning spells greater community efficiency, and from this greater efficiency, result important economies of time and money, thus affording both more money and more leisure.

Assuredly, there is no occasion to emphasize here the tremendous gains in efficiency of life in a metropolitan community, which accrue from a comprehensive, well-organized plan for the whole area and in which the component districts are set apart for their respective dominant functions of residence, business, industry, and recreation; each such district is not just conventionally but really functionally planned with respect to sizes and shapes of blocks, as well as the continuity and cross-section of local streets. The whole is tied together by a comprehensive system of main thoroughfares—radial, diagonal, and circumferential. Nor is it necessary to dwell on the colossal economy from such efficient, comprehensive, functional planning. Nevertheless, it is important to note that, from these gains in efficiency and economy, there must come to the great masses of the people greatly increased opportunities for culture. For although, of course, neither increase of wealth nor added leisure necessarily leads to increase of culture, both greatly enlarge the opportunities for some of its highest, most precious forms. Nor is there need to

urge that comprehensive, functional planning in itself produces the highest order of functional or organic beauty, the sort of beauty that results inevitably from the perfect adaptation of the form of an object to the use for which it is designed. All are familiar with the classic example of this: The clipper ship which, in the old days, was built for speed and the profit accruing from quick passages to foreign ports rather than for beauty, but which offers one of the most beautiful forms ever created by man.

Indeed, one does not have to go back to that time. The modern world is full of examples, such as the electric locomotive, the functional highway, the modern factory building, and the airplane.

Beyond this sort of beauty, there is still needed, through all planning and zoning, a conscious seeking for outward visual beauty of form, color, texture, and composition, such as can be supplied only by those specially trained to create it. While the mass of the people cannot be expected to unite readily in securing this beauty, it can be brought about through the leadership of the competent. When, through competent far-reaching regional planning, beauty in regional environment shall widely prevail, will not the people who grow up and work and play amid these beautiful surroundings, be far more intolerant of all that is ugly than people are to-day? Will they be satisfied any longer with what is merely poor, mediocre, indifferent, uninteresting? Does not beauty beget beauty?

Is it not true, for instance, that, if or when Philadelphia, or New York, or Boston, or Chicago, shall realize a wide, comprehensive dream of beauty, the influence over the country and over the world for beauty will be beyond one's utmost power to imagine?

The more perfectly organized the physical plan of the area of a community for health, efficiency, and economy, the more the members of that community can accomplish within it in a given time by the same expenditure of effort or money, and so the more time they will have over for cultural pursuits—pursuits which both further culture in themselves and also indirectly increase cultural opportunities.

This, however, is not enough, because a regional plan is not truly efficient that does not assure the absence of ugliness and the presence of the maximum of possible beauty throughout its whole three-dimensional extent, from dignified uplifting building groups to the artistic design of street name plates and mail boxes. No such dream as this can be realized that is not based on comprehensive, systematic, unified planning of the region. Then, out of this realized efficient and beautiful regional environment can come, and will come, more freely than ever before and far more widely, opportunities for culture.

NOULAN CAUCHON,* ESQ. (by letter).†—Engineers should have as an indispensable part of their education, a serious grounding in the principles of design in art. Without it, it is not safe to allow them at large. Look at the resultant atrocities of the merely utilitarian which some of them have perpetrated on the unsuspecting democracy. Henry Fairfield Osborn makes clear

* Ottawa, Ont., Canada.

† Received by the Secretary, October 16, 1926.

in his recent trenchant restatement of the truth of evolution* that the physical evolution of man is the least distinctively human part of him. In his intellectual and spiritual side lies the great advance beyond the brute. The brute has all the adaptabilities for fitness to purpose in a given environment. Are people to be restricted to thinking in merely the three quantitative dimensions of which the so-called "practical" man (and the brute) is cognizant, or should they not soar in a fourth qualitative dimension of the intellect and the spirit and insist on events and intervals being acceptable on the more human basis—something better than brute bluntness to impressions of hunger and shelter?

As Mr. Crawford so well puts it, things beautiful cost no more than ugly expressions of buildings, bridges, and landscape. Let planners express Nature, which is Truth, in its myriad forms, in such a way that all, from childhood to maturity, may get close to it in school, playground, and park, in home, workshop, and council chamber, to draw strength and cheer and guidance on the path of progress, individual and collective—National. Also, civilization has advanced to the point where amenity has become a negotiable value to the most dense. An art jury is desirable for all communities as a source of helpful, constructive criticism and restraining influence on private and public aberrations.

A further thought which Mr. Crawford advances for consideration is that public amenities, parks and parkways, should come into the daily life of the people, be used to and from, between their homes and work, and that they be planned to create this condition. Some years ago, it was the writer's privilege to suggest that in a mountain park development the pedestrian pathways, in threading their way through the park, should be kept as far as possible away from the highways. Need one argue with trained minds on the cultural opportunities that lie within regional planning and of the necessity that it be comprehensive planning?

It is the civil power which is always "skinning" the engineer's estimates, generally leaving very little with which to develop things of beauty.

Where Mr. Crawford attacks the gridiron plan he has, in the writer, a staunch supporter. The gridiron is vestigial of prehistoric limitation to two-dimensional thinking; it has an inherent property of congestion. On the other hand, hexagonal planning, in a four-dimensional state of mind, has an inherent property of diffusing traffic. The plan, by analogy, must fulfill the principles of organic evolution, that is, be adaptable to circumstance, that its continuity must be nourished by fitness for purpose—the high purpose of good living conditions and the fulfillment of human life.

E. M. Bassett,† Esq.—Houses will back upon parkways unless there is an intervening street. A parkway has no street front because the owner of the parkway (city or State) can build a high wall or put up an iron fence along its edge. The reason is that a parkway is a park. The abutting owners along a parkway have no easement of access or of light and air over a parkway. It is well to keep that distinction in mind between a highway and

* "Science and Religion in Education."

† Counsel, Zoning Committee, New York, N. Y.

a parkway. Parks should be zoned by municipalities for height and bulk. They can not be zoned for use, because all parks are devoted to recreational use, and only those buildings can be erected that are connected with recreation. If a school board begins to put up a school on a park area, any taxpayer can enjoin the board because it would not be a building intended for park use. In New York City, parks are zoned for height and bulk, but not for use. The zoning of a lot for a park would be wiped out by the Courts as soon as the property owner brought it to their attention.

Art juries can pass on beautiful designs for public buildings, and for buildings in streets and parks, but not for those on private property; at least in the United States. The reason seems to be that their's is the realm of æsthetics, and police power does not extend to æsthetics. Probably this is because the Courts conceive that they would have to depend on opinion evidence as to what was and what was not beautiful, and opinion evidence would vary so much that there would be no criterion. One expert might say that in a certain place a certain color was right and another would be equally sure that it was not right. In other words, the Courts in this country conceive that they cannot decide disputes regarding taste, and they look on æsthetics as a matter of taste.

However, it is proper and lawful to bring in the principle of design in city planning. For instance, the pyramidal buildings in Greater New York, that many architects like, are often considered beautiful. Part of their creation was city planning, and part was something else. They are mostly in the "two times" district on the zoning map; that is, the building can go up two times the width of the street on the property line, and then it must set back at the rate of one to four. The city planning part is to stamp on the land the zoning height district of two times. When the land is so stamped, it is for the architect to produce the building. If he would produce it in exact accordance with the set-back plans, it would probably not be a thing of beauty. It is the architect's ability to make a good design within the requirements stamped on the land that produces the beautiful building.

Jacob L. Crane, Jr.,* M. Am. Soc. C. E. (by letter).†—Writing from the viewpoint of the professional town planner and referring to the statement in Mr. Crawford's interesting paper that the civil engineers had the city planning field entirely to themselves for the whole of the Nineteenth Century,‡ it is true that the planning of streets and sub-divisions was carried out during that period largely by engineers and surveyors. However, the writer feels that "the stale, flat dullness" of "Main Street" towns and the "drab, stupid rectangularity of the checker-board cities" must be charged equally against the architects and builders who were responsible for the design and placing of the principal visual features, namely, the structures, and still more to the uncritical taste and frequently greedy haste of the American public.

The placing of the blame is not important except to realize that it is not only the engineers who need to be roused out of the easy habit of common-

* Municipal Development Engr., Chicago, Ill.

† Received by the Secretary, October 16, 1926.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1529.

place, hurried design. Further, the writer wishes to take exception to the reference to the "money-making power of municipal attractiveness". He believes that there is good reason for not continuously harping on the slogan that "beauty pays" in the campaign for a more satisfactory visual environment. As Mr. Crawford points out, the ugly ground plans and buildings are more rapidly replaced, while the beautiful objects continue to satisfy and are preserved. On the other hand, for the individual commercial enterprise, the ugly ground plan or building sometimes is actually most profitable to the individual concerned. In the second place, whether "beauty pays" is not the question. A visual environment, satisfying because of its loveliness, must be considered a justification in itself aside from its purely incidental dollars and cents value. Finally, the writer believes that planners can afford to be careful in their conception of what beauty really is. It has been easy for the zealous landscape architect to destroy the native picturesque beauty of a piece of marsh land, or a finely modeled hillside, or even an old abandoned gravel pit, by imposing upon them the neat and sophisticated design of the professional; and for the architect to spoil the structural feeling and silhouette of a building by too much ornamentation and, particularly, by too great ostentatiousness

The writer finds that some steel bridges, industrial plants, grain elevators, and machines are among the most beautiful things created in modern life. And they are all the work of the engineer.

Referring to Mr. Crawford's suggestion for "Bureaus of Building Advice",* it should be pointed out that such bureaus are, in effect, established in a number of towns and cities in the Middle West. They are constituted as special citizens' or architects' committees. They operate largely by persuasion, but with a marked effectiveness.

Replying to Mr. Crawford's criticism that zoning plans do not indicate sites for parks,† a legal zoning map cannot designate property for public use in advance of its having been taken for that use; and it is necessary to designate the zoning of all private property, even if some of it is recommended for park purposes on the general development plans, where such recommendations should properly be indicated.

The writer doubts the effectiveness of the suggestion‡ that golf courses may be made permanent open spaces by remitting the taxes on daily fee courses. Such courses are ordinarily privately owned by an individual or a small syndicate as a purely commercial enterprise; and, while the remission of taxes would undoubtedly be appreciated by such a syndicate, they could not be expected to hold the land for semi-public use when the opportunity arrives to profit by a large increase in land value.

There is a cultural opportunity in regional planning which may be worth mentioning, namely, the extension of the citizen's imagination to include in his civic ideal, not only the territory within the legal limits of his city, but the entire interrelated and interdependent region. It is not easy to define a

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1526.

† *Loc. cit.*, p. 1527.

‡ *Loc. cit.*, p. 1530.

cultural opportunity, but in all city planning and regional planning work one of the greatest cultural effects seems to the writer to be the stimulation of the imagination and the broadening of the interests of the average citizen.

Mr. Crawford very well points out that the dreamers are entitled to the credit for most of the advancement, and one of the primary purposes of regional planning should be to make a dreamer, not only of the engineer, but of all members of the community. Great city and regional plans will eventually be the crystallization of the dreams of many men and women.

Charles Wellford Leavitt,* M. Am. Soc. C. E.—The author has said much with which all can agree without hesitation, such as the dignity of elms bordering streets in a New England village engendering culture. In one such village where the elms were cut down to provide greater width for a street instead of building a parallel relief street, the people will regret it for the next 200 years at least, if not longer. The dignity and beauty of the town are gone, and it is now ordinary, sordid, and like many other ugly towns throughout the United States.

Art commissions and juries are important, with Bureaus of Building Advice on the side; the value of parks, parkways, playgrounds, and auto-parking places are admitted without argument. Zoning is good and should include zoning for parks. Putting a park between the industries and the home is fine. Central and branch parks with connecting parkways undoubtedly is a great thought. Art museums are certainly cultural with or without the central structure and branch attachments; and a central university with little universities all around sounds like the acme of cultural phenomena.

The Philadelphia Sesqui-Centennial with all the art removed would still be the best advertised exhibition in the world—by a prize fight. It was wonderful, if not cultural.

What is meant by a "poor topography" is a question. Topography is topography. Engineers have to take it as they find it and do their best with it.

Golf courses are certainly a God-send in providing some light and air. A good meal is pleasant to remember, but not to be compared with a vista, or a tapis vert—if one is not hungry. Throwing private golf courses open to the public by the payment of a green's fee might cultivate the masses; but would it be cultural for the club members? It is doubtful. Yet the speaker would like to see all golf courses maintained as open spaces in the future. Should golf courses be exempt from taxes? In case the course be sold for profit or sub-divided, the total accumulated taxes with interest should be paid at the time of transfer.

Traffic study is cultural, for without this cultural activity people would soon be dead, not cultivated.

A city or regional plan is apt to be held up if an attempt is made to carry on as a whole, or in a full and comprehensive way. The speaker is heartily in accord with the idea of knowing what has to be done, but of not advertising the whole plan at once, simply one of the items to be accomplished, and when this has been secured to take up others until the whole city has been re-made,

* Civ. and Landscape Engr. (Charles Wellford Leavitt & Son), New York, N. Y.

no matter how long this may take. A complete comprehensive plan is very apt to scare the taxpayers, the would-be developers of cities.

The city planner must be prepared to change the plan and keep it up to the best interests of the city, as ideas change frequently and very often for broader and better living conditions.

Does Mr. Crawford suggest that the decision be whether to speed or not to speed? If so, it will undoubtedly be "speed"; but speed made safe by, and with, culture. Probably the American will use the moments saved to go to a cultural movie.

The Fairmount Parkway was a dream. Through the persistent efforts of Mr. Crawford and others in Philadelphia, it was brought to a reality—a great achievement. The process of endeavor and achievement meant much culture. The parkway may still further cultivate if the city is governed by cultured officials. If not, the reverse may obtain, and this "culture" will be spelled with a "K".

No one has a greater respect for the designing talent of the architect than the speaker. It is regrettable that more engineers have not had greater training in the art of design and the sense of proportion which are so carefully studied by the architect. Some civil engineers, like some architects, landscape architects, city planners, lawyers, doctors, and plumbers, are not fitted for the job of designing a bridge, or a chair. However, the speaker must beg to differ as to the engineer being entirely lacking in the power to design. There have been many statements such as this. Consider the Brooklyn Bridge, and other bridges in United States parks, in Spain, and in Italy, the Washington Monument, the Sixty Wall Street Building, New York, the engineering part of the Delaware River Bridge; and then, by comparison, notice some of the later East River bridges and their approaches in which the architects have had a hand. The public buildings in Philadelphia, the New York Post Office, and the Capitol Building in Albany, N. Y., are products of architects—it does not require more than amateurs to judge that, in these instances, the palm will fall to the credit of the engineer.

It matters not whether he calls himself an engineer, a landscape architect, a city planner, or an architect, if he but have the power of design and sense of proportion. An artist versed in the sense of proportion and design can easily be recognized by his work. It is deplored that many who write titles after their names, meaning designers, fall far short of being true artists.

If its leaders are cultured, America may fairly compete with, or surpass, in the development of its coming race, the best examples in Europe of cultured citizens and resplendent cities. If we are not able to recognize true artistic leaders, we will be following false prophets and the development of our peoples and the appearance of our cities will fall far short of those ancient Greeks and Egyptians of whose achievements young students love to cant. We should follow the artist, whether he be a Brooklyn Bridge engineer, a Major L'Enfant, or the architect of New York's beautiful City Hall—John McComb; and we should avoid the designer or planner who, though he may call himself what he will—architect, engineer, or otherwise—cannot produce anything

better than some of the existing public structures which do not please the best sense of proportion.

By being critical and judging these objects for himself, whether he is professional or amateur, and by his efforts of seeing, observing, and careful analysis and criticism, an engineer will surely develop culture spelled with a "C".

MORRIS KNOWLES,* M. AM. SOC. C. E.—An apt illustration of culture in engineering is the Philadelphia-Camden Bridge. It is certain that there never could have been any controversy between the architect and the engineer on that bridge, else the result would not have been so successful as it is a great tribute to the profession and a notable example of the application of simplicity of engineering design. It is a production with which every one is satisfied.

After all there really is no need of a controversy. There are engineers who are good designers and also those who are poor designers, and there are architects in both classes. It is not a question of profession, it is one of personality; the criterion is whether the man himself has the ability to see beyond and through the things he is trying to do. It is not a question of whether he is an engineer or an architect, any more than it is a question of whether a lawyer can be a successful business man. Many engineers make successful business managers and many others cannot successfully handle their own affairs. Engineers do poorly to discuss this question from the professional angle. What they should do is to try and cultivate and stimulate culture in every one, be he an engineer or an architect.

HAROLD M. LEWIS,† M. AM. SOC. C. E.—Perhaps the reason both engineers and architects have fallen down, to a certain extent, in trying to arrive at the best solution of a modern city plan, is the great increase in the complexity of such plans. Mr. Harvey W. Corbett, a well-known New York architect, who has also had an engineering training, has said that the engineer deals in "facts" and the architect in "fancies", and that both are essential for the successful completion of a large modern building. The speaker believes this should be supplemented by saying that they both deal in design, based primarily on different points of view.

A combination of fact and fancy is still more essential in the development of a practical and adequate city or regional plan. The great size of large metropolitan centers has been made possible by improved methods of transportation, sanitation, and building construction. This has brought many new problems, with which the engineer is specially trained to deal. Carefully prepared and analyzed facts are necessary as the basis for any plan, but the successful regional planner must have broad vision and must be somewhat of an idealist. Engineers would do well to develop such qualities to a greater extent.

The future should not be merely an extension of past trends, but should bring opportunities for improved culture of the kinds referred to by Mr. Crawford. As the execution of a regional plan is based on a co-operation between local planning authorities, so should its preparation be based on the

* Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

† Executive Engr., Regional Plan of New York and Its Environs, New York, N. Y.

best that both engineers and architects can give. If such co-operation is successfully accomplished there is plenty of room for credit for both, where credit is due.

R. H. Randall,* Assoc. M. Am. Soc. C. E.—It may appear at first that Mr. Crawford is just a trifle hard on the engineer, and that the outlook is dark, or even black. Somewhere, perhaps in the explanatory text on the back of the U. S. Geological Survey standard maps, there is a phrase to the effect that "culture, or the works of man is shown in black". While it is good map practice to show the works of man (which are naturally and usually the product of the engineer) in black, the terms, "culture" and "black", are not synonymous, even by inference and as they are chargeable to the engineer.

Both engineers and architects have produced structures so ugly as to be actually limited in their usefulness. However, there is noticeable a present and increasing betterment in this respect. Mr. Crawford's paper should serve as a valuable incentive toward continued improvement in both engineering and architectural design; a reminder of what now and always needs to be done, rather than an arraignment for past shortcomings.

C. H. Howell,† M. Am. Soc. C. E. (by letter).‡—The author states§ that, "if an engineer builds a bridge merely sufficient to stand the traffic, he has not designed the bridge; he has only constructed it; he is only a plumber". However close to the plumber the construction engineer may be in Mr. Crawford's opinion, it is certain that some one designed the bridge. Presumably, it was a bridge engineer.

Webster's Dictionary defines the word, "design", as follows: "To produce a scheme or plan for the making of anything". Anything that is planned is designed, according to this definition, and Mr. Crawford is, seemingly, at variance with Webster.

The writer also submits, that the one outstanding example of non-rectangular city planning of this country, Washington, D. C., was the design of an engineer, L'Enfant.

Not all the "drab, stupid, stale rectangularity of the checker-board cities" should be charged against civil engineers. Some of the drabness, stupidness, and other terrors of the older parts of old cities are the results of, shall it be said, designs by architects? The writer believes the best engineering talent of the Nineteenth Century produced work comparable to that of the best architects and that the common or garden variety of engineering work was quite superior to the common run of architectural efforts.

The graduate, to whom Mr. Crawford referred, need have no fears about architects supplanting civil engineers as designers of bridges. Bridge design is a highly specialized science. No mere architect can afford the time to master it. If he does master it, he becomes an engineer. "The end is not yet"—except in dreams.

* Pres., R. H. Randall & Co., Toledo, Ohio.

† Designing Engr., Middle Rio Grande Conservancy Dist., Albuquerque, N. Mex.

‡ Received by the Secretary, September 10, 1927.

§ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1529.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

FORECAST: THE REGIONAL COMMUNITY OF THE FUTURE

Discussion*

By Messrs. Robert Kingery, Howard Strong, William J. Wilgus, Frederic H. Fay, and William T. Lyle.

Robert Kingery,† Esq. (by letter).‡—In Mr. Adams' splendid forecast of the regional community of the future, the writer finds the expression§ that is the foundation of the entire activities of the Chicago Regional Planning Association, namely, "regional co-operation will have ended the pollution of drinking waters and developed comprehensive systems of sewage disposal". In the region surrounding Chicago, city planning has had its "ups and downs" just as in many other parts of the country. A large number of city and village plans have been prepared, splendid reports have been written, and nothing has been done about them. Many of the local and public officials scoff at city planning and call it the impractical dreams of the "uplifters".

The reason for this is not difficult to find. In most cases city and village plans have been prepared with a minimum of co-operation and aid from the local city engineer and official who, under the law, is charged with the responsibility for spending the money which would be used for carrying out the project proposed in the plan. The second reason is that the apparent objective of the plan commissions is a plan and a report, and not so much attention is given to perfecting organizations that will put the plan into the construction stage.

The City of Chicago is the notable exception. A great body of citizens have been appointed on a permanent plan commission, and they have insisted upon maintaining a staff of engineers for the purpose of carrying out the Chicago Plan.

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* This discussion (of the paper by Thomas Adams, Esq., presented at the meeting of the City Planning Division, Philadelphia, Pa., October 8, 1926, and published in September, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Secretary, Regional Planning Assoc., Chicago, Ill.

‡ Received by the Secretary, October 16, 1926.

§ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1541.

Regional planning is far more difficult than city planning because of the greater number of local governing officials, each of whom has a specific duty under the laws of his State, and under the powers granted his particular office.

To proceed with regional planning in such a way that each of these public officials of highway commissions, drainage districts, sanitary districts, park districts, the private utility organizations, such as telephone, electric light, rapid rail transit, and others, has a share in the preparation of plans is to develop a plan or series of plans which, in advance, holds the confidence of the public or private officials who are going forward with their public works whether or not there is a regional plan.

In the Chicago Region, the Planning Association has thus far been moderately successful by combining, into a general highway scheme, the different programs of the many highway building agencies, simply by arranging that they meet over a common general map, study how their own programs fit, or do not fit, with those across the imaginary, invisible boundary lines. In practically every case these officials have agreed on such modifications of their programs as were necessary to produce a system of connecting highways.

The fact that six county boards, with a combined area of 3 150 sq. miles of unincorporated land, have adopted and are administering sub-division regulations which require the real estate men to meet certain high standards in the platting of land, indicates that the method used in drafting these regulations was sound. Members of the Sub-dividers Division of the Real Estate Boards, plat officers of counties and cities, the surveyors' organizations, and the city planning engineers combined in developing and agreeing on these regulations. They are in ordinance form also, and are rapidly being adopted in the cities and villages of the Region. If a little group had retired to the privacy of their offices to develop such a regulation and then had sought to put it into effect, without having had the benefit of all the views that went into its determination, it is doubtful whether it would have been in effect in any of the cities and villages to-day.

If the Regional Planning Association of Chicago can make one suggestion to regional and city planners, it is that regional and city planning is not alone a gift of a few men who have had certain training and have done certain reading on the subject. Some of the most obscure city, county, sanitary district, and park officials and some of the public utility engineers can do, and are doing, by co-operation in the Region of Chicago, some very successful regional planning.

Howard Strong,* Esq.—The author has given a masterly presentation of some of the evils that have resulted from modern concentration of population; of certain economic and social fallacies that have controlled the development of the modern city; and has presented an enheartening picture of what may be hoped for the future of New York and, by implication, of other large cities.

However, as he states, who can calculate the forces that will mould the city of the future? Who can predict the scientific discoveries, the overturning in social and economic thought that may revolutionize human action? A

* Secy.-Director, Regional Planning Federation, Philadelphia, Pa.

pattern for the future growth of the city may be moulded and complete, when some simple invention may upset the entire calculation.

In 1890, or thereabouts, an ordinance was passed in Cleveland, Ohio, requiring that every self-propelled vehicle on the city streets be preceded 100 ft. in advance by a man carrying, by day, a red flag and, at night, a red lantern. It may be assumed that the development of the horseless carriage to its present stage of efficiency, has rendered somewhat impracticable this particular application of the accepted principle of the responsibility of the community for the safety of its citizens. The provision of Bear Mountain Park, thirty-five miles from New York City, would have been preposterous twenty-five years ago, but the present widespread use of automobiles has made such a regional park the greatest of boons. The proposal of a large neighborhood theatre in every community of consequence, in those days, would have led its proposer to a court of inquiry, but the moving picture has made such provision an accepted necessity of village life. The complex dynamic forces which operate and the infinite potentiality of scientific discovery and of changing thought, may lead along any one of a thousand different avenues.

What possibility, then, is there for foreseeing and preparing for the future? Does not the greatest hope lie in the direction of discovering the underlying principles which must control the life of large numbers of people living closely together, in bringing a general acceptance of those principles and in relying on each generation, profiting by the experience of the past, to make its own application? For principles, if they are sound, are constant, however much their application may vary with the changing conditions. The Cleveland ordinance was one of the earlier recognitions of the responsibility of the community to protect its citizens against the dangers of community life. The application is now obsolete, but the principle is accepted and its application is being constantly expanded to new fields. If each succeeding generation can make some definite contribution in establishing these underlying principles, or in defining and charting the fields in which they apply, engineers will be assured, with the constant development of the science and art of planning, that the pattern of which Mr. Adams speaks will ultimately be found. It will be, however, largely a pattern of principle, the application constantly varying with changing conditions and demands.

What then, are some of the principles the acceptance of which must be secured or the application of which must be extended?

The police power, the principle that a man may do with his own as he pleases, only to the point where that use begins to endanger the health, safety, morals, or general welfare of other members of the community, is a principle written into common law, but its application is being constantly extended. The right to rob his neighbors of light and air, which was formerly claimed by the average citizen as a God-given right, is gradually disappearing under the expansion of this power; but recent adverse zoning decisions indicate that the application of this principle in its most complete sense is far from universal, and that its field of application is still more limited than some planners believe to be sound. Is it too much to hope, for instance, that the common right to beauty may some time be recognized under this power? Already the

regional planner is touching upon this field, as is shown in the recognition, in a few States, that a flaming work of art setting forth the attractions of "Wobbly's gum", or the "cigarettes of which father has intimate knowledge", or, for that matter, the Sistine Madonna itself, however attractive they may be on the magazine cover or over the piano, may be a violation of human rights when set forth on a billboard that hides a lovely vista or some noble monument from the passer-by.

Another underlying principle, which must ultimately attain complete acceptance, is the recognition that community-created wealth belongs, not to the speculator, but to the whole community. The reformer who seriously proposes this is still nominated a Socialist, but a beginning of its recognition is found in the procedure of excess condemnation and the assessment of benefits. Many look forward to a modification of this economic system that will assure to the community the unearned increment which it creates, and enable the land speculator to apply his vision and imagination to some activity more socially and economically sound?

A whole set of principles with reference to land occupancy and rentals is yet to be developed. Mr. Adams quotes Belloc in his distinction between imaginary and real economic values,* and hopes for the time when cities will generally recognize the investment values of public improvements. The recognition of the vicious circle created by the high building, with its increasing land value and its consequent necessity for still greater height, seems to point to the coming recognition of a principle the application of which may change materially the facades of buildings in the financial districts and demonstrate anew the rental value of the low building. Certainly, the principles of taxation, as applied to assessments on land and improvements, and their effect on centralization and decentralization are vaguely understood. The regional planner is looking for the evolution of a new or a modified principle of local government that recognizes the validity of a definite relation between the area of governmental control and the social and economic unity of the group. The present administrative units of large metropolitan areas are in no degree identical with their common social and economic interests. The recognition of this principle will surely result in some form of regional government or federation which will aid greatly in solving metropolitan problems.

There are many nominally accepted principles that need a new statement and a new acceptance in terms of the new civilization. Life, liberty, and the pursuit of happiness, meaning life and work under wholesome and inspiring conditions, the liberty of open spaces and free movement, and that happiness which can be attained only through opportunity for education and culture and full rounded self-development, is one of them. Other principles are being discovered, whose validity is only beginning to be felt, but whose full recognition and application as practical standards of human life, will go far in assuring to future cities greater social and economic service to coming generations.

* *Proceedings,* Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1537.

WILLIAM J. WILGUS,* M. AM. SOC. C. E. (by letter).†—No more important duty rests with the engineer, and none more fascinating, than pointing the way for the wise expansion of great centers of population. Critics very properly may say that the continued growth of cities is to be deplored, but the fact remains that this is a condition and not a theory. Modern opportunities for pleasure and profit in urban communities are causing human beings to be increasingly gregarious. To attempt to stem the tide will do no good. The engineer, in going with it, must do all in his power to alleviate its attendant evils and devote his gifts of vision and creativeness to forecasting the needs of the future and devising means for their attainment. This he must do while there is still time to prevent the unwise doing of things that later cannot be undone.

Mr. Adams has glowingly pictured the urban region of to-morrow as it should be; but to translate his convincing dream into the much hoped-for reality, two formidable obstacles must be overcome. They are (*a*) lack of centralized organization through which efforts may be co-ordinated and real results accomplished; and (*b*) a lack of capital with which to finance projects of enormous magnitude.

The first of these obstacles is serious because a multitude of varying interests, too often antagonistic or indifferent, have not found a common ground on which to approach the law-making bodies for needed legislation. The remedy would seem to lie in the voluntary co-operation of all interests in the regional community—political, corporate, and individual—by an organized method that would fairly recognize the weight that should be given to the voice of each member. Such a body, working through a small executive committee of high-minded, able, and public-spirited citizens, and guided by the advice of experts of note, should be successful in formulating plans and legislation acceptable to enlightened local sentiment and, therefore, compelling upon the legislatures and upon Congress. From a movement of this kind should come forth a compact organization having the will, the capacity, and the power to accomplish things for the general good.

The second of the obstacles would fade away if the idea could be dispelled that funds for public works necessarily must be provided through taxation. That is a bugbear that paralyzes the community and kills progress. The average citizen looks on taxes with aversion because he sees no tangible direct return for his expenditure. As a rule he does not object to paying for that which brings an obvious immediate reward that caters to his pleasure, convenience, or necessity, or by reason of which he may make or save money. A case in point is the Holland Vehicular Tunnel, beneath the North River, which has been financed by the States of New York and New Jersey on the promise of toll collections from those who will use the tunnel in preference to ferries. In this respect, the beneficiaries who pay the bill would have been even better served if real estate bordering the entrances and exits had been compelled to contribute toward the cost a fair share of the increase in value which it is reaping because of the changed conditions. Another instance is

* Cons. Engr., New York, N. Y.

† Received by the Secretary, September 30, 1926.

that of the Grand Central Terminal development, to which the writer ventures to refer with some diffidence because of his responsibility for its inception. There, departure from precedent and the utilization of dormant "air rights" have brought a golden reward from by-products much more than sufficient to carry the enterprise without placing an additional burden on the traveler or the public at large. In a word, lack of capital, as an obstacle to adequate planning and building for the future, will disappear if Man will have the ingenuity to utilize, to the last degree, all revenue-producing by-products that can be made to flow from the new conditions, and also the courage to allocate the remaining net costs among the beneficiaries, namely, the users and reapers of benefits.

Engineers, then, owe their fellow citizens, not only dream pictures of what should be done for their own and their children's welfare, but also forceful reminders of what they must do to realize them. These may be attained by organizing effectively for the adoption and fruition of sound plans, and by financing new projects in part through the utilization of by-products and in part through the allocation of the remaining costs among the beneficiaries rather than exclusively through taxation foisted on the public at large. Stated differently, as the writer sees it, effective organization and self-support to the fullest possible extent, are the keynotes of success in the timely fulfillment of major plans for the future of regional communities.

FREDERIC H. FAY,* M. AM. SOC. C. E. (by letter).†—This excellent paper brings strikingly to mind the rapidly widening field of city planning and the need of ever-broadening vision on the part of those who have to deal with the future development of urban communities. It is only a little more than thirty years since the World's Columbian Exposition at Chicago, Ill., awakened an interest in the question of the development of cities. In the popular mind, city planning in this country dates from the Middle Nineties, and at first dealt essentially with small units within the cities and with matters of appearance, such as the placing of public buildings, the development of civic centers and open areas, etc. City planning was then synonymous with the "City Beautiful". A few years later consideration came to be given to the orderly planning of the development of cities as a whole, and while attractiveness was still an important element, emphasis was laid more especially on the welfare of the people and the conditions under which they live and work. It is only a few years since, for the larger cities, that planning has been carried systematically beyond the boundaries of the municipality and that consideration has been given to the needs and the development of the larger territory, which will be a part of, or within the zone of influence of, the larger city of the future. With the increase in area within the scope of city planning comes also increase in the number and diversity of problems falling within the scope of the planner's consideration. Before many years, properly constituted agencies, systematically studying and planning for the development of States and of the Nation as a whole, will be common. When that time comes some

* Cons. Engr. (Fay, Spofford & Thorndike), Boston, Mass.

† Received by the Secretary, October 16, 1926.

broader term than city planning must be adopted to indicate the field of the planner's activity.

New York presents problems in regional planning that are doubtless as large, diverse, and complex as those of any great city of the world. Other cities of lesser size, however, present problems in regional planning on scales smaller than New York, but often with diversified features. In fact, each large urban region has planning problems purely its own, although many problems are fundamental and common to all.

The Boston Metropolitan District is an urban area in which regional planning along certain lines has been carried on for many years through force of necessity. In fact, the Boston Metropolitan District is probably the earliest example, in this country, of official action in regional planning. The District to-day comprises, in addition to the City of Boston itself, thirty-nine other municipalities.

Table 1 is a comparison of the areas and populations of the City of Boston, the Boston Metropolitan District, and the City of New York. It is interesting to note that in 1925 population density per square mile within the city limits of Boston was 16 306, as compared with the corresponding figure of 18 796 within the city limits of New York, Boston thus having a density about seven-eighths that of New York City.

TABLE 1.—AREAS AND POPULATIONS OF CITY AND METROPOLITAN DISTRICT OF BOSTON, MASS., COMPARED WITH NEW YORK, N. Y.

Districts.	Area, in square miles.	Population in 1925.	Density, in 1925, per square mile.
City of Boston	47.8	779 620	16 306
Boston Metropolitan District	409.3	1 808 845	4 419
City of New York	299.0	5 620 048	18 796

What is the story of regional planning in the Boston Metropolitan District? Massachusetts was the first State in the Union to establish (in 1868) a State Board of Health. About that time the City of Boston and other near-by municipalities took steps to improve sanitary conditions along streams draining into Boston Harbor, by the construction, independently of each other, of certain sewerage and drainage works. The territory was so closely built upon, however, that it was soon realized that independent action by the several communities must prove ineffectual. The State Board of Health took a hand, and the result was the establishment, by legislative enactment in 1889, of the Boston Metropolitan Sewerage District and the creation of the Metropolitan Sewerage Commission to handle the sewage and drainage problems of the district as a whole, irrespective of municipal boundary lines. This is believed to be the first official step taken by any State in regional planning. In 1893, the State Legislature created the Metropolitan Parks District under the charge of the Metropolitan Park Commission, which in the succeeding years, has admirably supplemented the nucleus of the great park system previously

established by the City of Boston, such that now the park areas of the Boston Metropolitan District are widely and favorably known throughout the country. In 1895 the necessity for co-operation between Boston and the surrounding cities and towns in the matter of water supply led to the creation of the Metropolitan Water Board and Water District, also by State legislation. More recently, the maintenance and operation of the Metropolitan Parks, Water Supply, and Sewerage Systems has been under a single commission, the Metropolitan District Commission. However, in 1926, the necessity having arisen for a large addition to the Metropolitan Water Supply System, a special commission was created by legislative enactment to develop a system of additional water supply.

The regional planning of the Boston Metropolitan District thus sketchily outlined, has been that relating to certain specific problems in which the mutual interests of the cities and towns of the District forced co-operative action. In each case this regional planning and the constructions resulting therefrom, have been carried out by an official State Commission.

More recently, in the Metropolitan Boston Region, a strong sentiment has developed regarding the need of regional planning on broad lines; planning that is not limited to a few immediate and specific problems, but looks ahead over a long period to the general development of the city of the future. Largely through the initiative of the Boston Chamber of Commerce, there was created by legislative enactment in 1923 the Division of Metropolitan Planning within the Metropolitan District Commission. The Metropolitan Planning Board consists of seven members, four of whom are State or City officials representing separate departments, and three, including the Chairman, are appointed by the Governor from among the citizens of the District. The Metropolitan Planning Division was created primarily to study the question of transportation in its broadest sense in the whole metropolitan area, transportation by rail, water, highway, and air. The problem of transportation, however, is necessarily linked with and dependent on other problems of metropolitan development such that the field of activity of the Metropolitan Planning Division becomes a broad one. The Division is given a reasonable appropriation for its work, and it has a permanent engineering and technical staff. Therefore, while in New York the great problem of regional planning is being carried out by private agency, in Massachusetts many of the broad problems of regional planning for the Boston Metropolitan District have been and are being studied and solved through official action by the State.

The question may arise, "Why the need of regional planning?" In the case of Greater Boston, for example, where the cities and towns of the Metropolitan District having common interests are solely within the limits of a single State, why not annex these cities and towns to Boston itself and thus establish as a single municipal entity the real city of Boston as a single municipal unit? Then all problems of regional planning could be dealt with directly by the government of the larger city. This has been often suggested and as often rejected. It brings up an interesting point of city and regional planning, that is, the psychological feature. In the case of many of the smaller cities and towns surrounding Boston, local pride, historical tradition, and differ-

ences in racial and political complexion, all weigh against outright annexation. The communities are jealous of their existence as separate entities, although willing to join with others in matters of mutual concern. It seems unlikely that "Real Boston", as many people call it, will result from the annexation of approximately forty municipalities of the Metropolitan District, the interests of which are to large extent identical with those of Boston. On the other hand, these various communities are gradually being knit closer together through common bonds of interest, and the best development of the real Boston of the future can take place only through wise regional planning which takes account, not only of the physical, but also of the psychological problems involved.

What the Boston regional community of the future will be is largely a matter of speculation, although its development along certain lines may fairly be forecasted through economic necessity. As Mr. Adams well states,* "There is no true economic basis for the congestion of great cities like New York, and if it is continued the effect will be to strangle the pulse center of the community." The work now under way in planning for the transportation needs of the future with the increased use of the automobile, will do much toward dispersing population and the development of suburban areas. The growth in population, therefore, of Greater Boston is likely to be almost entirely in the territory outside the present city limits. The Boston Metropolitan District offers a broad and inviting field for regional planning which, if well done, may be of inestimable benefit to those who are to live in Greater Boston in the next and succeeding generations.

William T. Lyle,† M. Am. Soc. C. E. (by letter).‡—The remarkable progress made during the last few years, and even during the last few months, in city and regional planning is attracting wide attention. This progress is not so much in the art itself as in the confidence of the public of its possibilities, a confidence promoted by an expanding conception in the minds of city and regional planners of the scope and dignity of their art. City and regional planning is assuming an aggressiveness not characteristic of recognized conservative engineering.

This paper is suggestive and stimulating. In city and regional planning the writer would call attention to the folly and even futility of incomprehensive work. The art must operate not only in its physical, but also in its human applications. The older planning was concerned with maps, layouts, and architectural developments; the new planning is considering human beings in their business, domestic, recreational, cultural, and even moral requirements.

City and regional planning needs both vision and courage. Its determinations are not always made on the basis of well buttressed predictions. It is doubtful if they can be, with the determining factors of transportation and power development in a state of uncertainty and change. City and regional planning constitute a new field of engineering—broad, comprehensive, dignified—reaching out into the realms of economics and human relationships.

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* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1537.

† Prof., Civ. Eng., Washington and Lee Univ., Lexington, Va.

‡ Received by the Secretary, September 30, 1926.

The new planning calls for promptness in execution. When a recognized need is experienced there should be a corresponding response in construction. In public works, however, a lag almost always occurs between the need and the fulfillment. Of course, exceptions may be cited to the rule. Some communities are more far-sighted, courageous, and energetic than others. In projecting and constructing its new water supply the City of New York furnishes a good example. The new work was carried through before the dreaded succession of dry years became an actuality. So, also, in its response to the challenge, "Shall we save New York?", it passed the zoning ordinance of 1916, which has wrought such a benefit in the stabilization of real estate values.

Regional planning to-day is applying the principles of harmonious relationships so important in business. Since their establishment, American cities have grown without regulation. They are notorious examples of discord, where selfish interests in private promotion schemes were permitted to operate without check and were even supported by law.

The unregulated liberation and production of power in modern civilization may constitute the gravest menace. Only when properly directed and utilized can it become a blessing. Prosperity will result in the even balance between a healthy need and an adequate and well-directed supply.

The author considers that "perhaps the most hopeful fact in the metropolitan regions of America is the extent to which land is being acquired for parks".* Proofs of this are easily established, not only on hygienic, cultural, and moral grounds, but for economic reasons as well. The assessed valuation, in 1856, of the three wards adjoining Central Park, New York, was \$20 500 000. By 1873, it had risen to \$236 000 000. The natural increase as obtained by averaging the gain in the other wards was found to be \$53 000 000, making the earning capacity of the park during seventeen years for the three adjoining wards, \$162 500 000.

City planning heretofore has emphasized too much the visible to the neglect of the invisible. It has concerned itself with streets, parks, railroads, and civic centers, and has neglected drainage and water supply on the assumption that they are properly and exclusively in the province of the city engineer. This should not be so, for drainage and water supply powerfully influence the selection and improvement of parks and parkways, and, in turn, are equally influenced by them. The foul drainage channel by reason of its location and the architectural asset of trees and sloping banks is the ideal location for a parkway; and the great forest preserve or reservation the ideal location for a reservoir.

Much has been written on the subject of overcrowding and congestion. The companion idea of undercrowding deserves more thorough consideration. The fundamental law is the fullest and best use of each unit of area, depending on its location and other characteristics. Less need be said than formerly on the debated question of concentration *versus* decentralization. The

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1535.

future regional community will derive its character from the spreading of metropolitan populations.

In the new regional planning, governmental barriers that have restricted progress will be broken down. For example, a great river, the Delaware, the upper reaches of which have never been extensively utilized for public benefit, will be harnessed for power, and will be drawn from as an important metropolitan water supply. This discussion does not contemplate the consideration of political questions, although the writer is convinced that political doctrines that continually challenge human progress will eventually give way.

Mr. Adams speaks of great centers like New York and Philadelphia, Pa., striving to maintain supremacy in competition with the surrounding urban regions of the future.* This may be the case, although it is to be hoped that wiser counsels will prevail in the development and perfecting of controlling doctrines of city and regional planning, namely, co-ordination and co-operation. The surrounding regions need the centers and the centers need them. Neither can fully prosper in adverse rivalry with the other. Subsequently, however, he speaks of the regional co-operation needed to end the pollution of drinking waters and promote the development of comprehensive systems of sewerage.† It is this co-operation that is so necessary and that is so replete with rich reward. An example of the benefits of co-operation is to be found at Boston, Mass., where several municipalities, each guarding its local autonomy, have wisely united to obtain the benefits of a water supply, sewerage system, and parks which would be beyond the reach of each municipality operating independently. A great metropolitan region will come into its own as fast as it learns, practically, the important lessons of harmonious co-operation.

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* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1540.

† *Loc. cit.*, p. 1541.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

EARTH WORK BY THE HYDRAULIC METHOD

Discussion*

By DeWitt D. Barlow, Assoc. M. Am. Soc. C. E.

DeWitt D. Barlow,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author has described a method for classifying field data with the view to building up a store of information that will enable dredging engineers to estimate their work with less uncertainty. The proposed method is to record the hourly "capacities" of hydraulic dredges wherever they work in uniform beds of material, and simultaneously to record certain physical characteristics of the material as the same may be revealed by laboratory tests for absorption, flocculence, etc. The method seems obvious enough, especially to one not acquainted with the physics of the problem. It assumes that the output of the dredge varies with the previously mentioned characteristics of the material dredged, and with nothing else. Both assumptions are false.

In the writer's opinion the procedure described under the heading "Proposed Method of Applying Data Secured",§ will not produce results of any value. The capacity of the dredge is a variable quantity of great complexity, contingent on many things other than the material dredged.

The output of a hydraulic dredge (per engine-hour) is governed by two main factors, both of them complex; that is, the amount the dredge can dig, and the amount it can pump ashore. Either quantity may control. It cannot pump more than it can dig, and it cannot continue to dig more than it can pump, so that the first thing the experimenter must do, is to classify the material as to its digging and its pumping qualities.

The digging must be canvassed, not on the basis of what the dredge can do, but on the type of cutter used, the horse power and torque applied to the

* This discussion (of the paper by Roy E. Miller, M. Am. Soc. C. E., presented at the meeting of the Construction Division at Seattle, Wash., on July 15, 1926, and published in September, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Pres. and Director, Atlantic, Gulf & Pacific Co., New York, N. Y.

‡ Received by the Secretary, September 20, 1927.

§ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1717.

cutter shaft, and the tension and horse power on the swinging wires. When these data are assembled, they should be examined in connection with the physical characteristics of the material with which they vary.

The author's statement* that "more often than not the ability of the dredge to excavate the material determines the output, rather than its pumping capacity", is only another way of stating that the dredges that have come within his observation were under-powered on the cutter or over-powered on the pump. It is incorrect.

The best way to classify material with respect to cutting is to measure its resistance to cutting. For this purpose the writer prefers an auger to any amount of litmus solution. Entirely satisfactory estimates are made in this manner. The hazards in hydraulic dredging estimates do not reside there. They are to be found in the extreme sensitiveness of the pumping operation to differences in the coarseness of particles to be pumped and to variations in the material excavated from the samples on which the estimates were based.

Consider the question of pumping, assuming that the cutting power is adequate. The operation of hydraulic dredging consists in maintaining a flow of water through a pipe line and of introducing into the stream, solid material which is carried in suspension and along the bottom of the pipe. The cutting or digging, referred to, is the introducing process. The stream is kept in motion by a centrifugal pump; the solid material is introduced on the suction side. In the conduct of the operation it is possible to run more solids into the stream than the stream will carry off. That is what is meant by the statement that the pumping controls. Under that condition some of the solids settle out and lie in the bottom of the pipe, the cross-section is throttled, the output is diminished, and a limit is found to the pumping capacity. This limit is controlled by many factors; among others, the available torque on the pump, the maximum speed of the pump, its efficiency (which varies from zero through 70% and back to zero for different speeds and heads), the horse power of the prime mover, its power characteristics, the length of pipe line, its size and character (pontoon lines ordinarily consume, in power per unit of length, from 150% to 200% of that consumed by the shore line), the elevation of the end of the hydraulic grade, and the material.

In order to discover the effect of variation in material on the rate of pumping, it is necessary to eliminate or standardize the other factors. The way to do that is to ascertain, not what the dredge will do in a given material, but to determine the horse power (hydraulic) consumed per unit length of a standard pipe, per cubic yard of that material carried in suspension at the saturation point. That figure is important and is the one that dredging engineers want. It varies with several factors, more especially specific gravity and coarseness. Most sands, clays, and muds are of a silica base so that the specific gravity is usually constant. There is, of course, the widest range in the size of particles, and a corresponding wide range in the "dredging power factor", if the writer may be allowed to coin a term. In fact, the "dredging power factor" is so sensitive to the size of particle that hardly anything else

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1718.

counts. As between fine beach sand and sand of the consistency of granulated sugar, there is a range of 300% or more. If one goes as far up the scale as gravel of, say, the size of lima beans, the range will be more than 1 000 per cent. The larger the particle, the more power required to move it.

The way to determine the coarseness of material is to sift it. When that is done and its specific gravity is known, one can classify it as to its power factor, but the error, due to the character of the experimental data, will be a quantity of far greater magnitude than the whole field affected by capillarity, and the experiments mentioned by the author.

These remarks should not be taken in disparagement of research. It is badly needed in dredging. The writer is enthusiastic for research and wants to direct it where it will be effective. The author's method is far too empirical. It assumes a simple relation where there is a complex one, and the aspect of that relation which it investigates is negligible. It is completely masked by the major phases of the problem, which he ignores.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

PROBLEMS IN CONCRETE DAM CONSTRUCTION ON THE PACIFIC COAST

Discussion*

BY MESSRS. ARTHUR P. DAVIS AND M. M. O'SHAUGHNESSY.

ARTHUR P. DAVIS,† PAST-PRESIDENT, AM. SOC. C. E.—The speaker agrees with the stated undesirability of manufacturing sand where it can be avoided. At the site of the Roosevelt Dam, for instance, no suitable sand was available in Nature and had to be manufactured from existing materials. The first attempt was with a pure hard limestone which was being used in the manufacture of the cement. In crushing this rock for sand it had a tendency to chip into long thin flakes and was deficient in fine particles. After some experiments with that, an attempt was made to mix it with other rock, and, finally, a mixture of half limestone and half sandstone was adopted, both of which were conveniently at hand and made an excellent combination which tested high and was satisfactory throughout the job.

Another experience was that of the contractor in the construction of the Shoshone Dam, Wyoming, where conditions were especially difficult on account of limited working space, a narrow gulch with high cliffs on each side forcing such operations some distance up stream where more space was available. No suitable sand was available in sufficient quantities, and the little that was at hand required washing and was very fine. The contractor tried to crush granite rock which was very hard, and this hardness made the operation extremely difficult. One machine after another was worn out and cut to pieces in use. It was almost impossible with that granite to get a sufficient proportion of fine material without discarding a great deal of coarse. Finally, by washing the fine sand and using this, to the extent available, with the

* This discussion (of the paper by Arthur S. Bent, Affiliate, Am. Soc. C. E., presented at the meeting of the Construction Division, Seattle, Wash., July 15, 1926, and published in September, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Engr., East Bay Municipal Utility Dist., Oakland, Calif.

granite, it was possible to make a passable sand still lacking somewhat in gradation.

Another experience was in the Klamath Valley, Oregon, where no rock was available except lava flows. After a great deal of experimenting, it was discarded and sand was shipped from Marysville, Calif. After these and other trials, the speaker is certainly in accord with the motto of the author, "don't do it".

M. M. O'Shaughnessy,* M. Am. Soc. C. E. (by letter).†—Having been identified with the construction of about twenty dams, the writer is sincerely interested in some of the viewpoints advanced by Mr. Bent.

All first-class engineering plans should include information about topography, water run-off, high-water period, weather history, water supply for the entire year, length of working season, and possible methods of river diversion. It should not be necessary for the contractor to waste any time in worrying over those subjects, as any one who is going to build a large dam must have employed engineers of sufficient standing and experience to procure precise information on all those subjects.

In the matter of excavation for foundations, careful consideration of surface examination may disclose, when the digging is done, that the bottom will be quite different from that anticipated, and it is well always to provide a liberal estimate for such contingencies, which are bound to occur.

Experienced rock men can blast a foundation without serious damage to machinery, if the latter is properly placed. Except in the core trench and scab rock on the surface, the shooting should not be serious. Cleaning the bed-rock is an important element. A successful expedient in the foundation of the dam at Hetch Hetchy was the use of a sand blast to clean off the old scum from submerged granite rock, which was buried under 100 ft. of loose rock and gravel.

Mr. Bent is correct in stressing the necessity of a railway.‡ A standard gauge railway, 68 miles long, was built to the dam at Hetch Hetchy. Six locomotives were found to be inadequate at the peak period of construction, and a seventh locomotive had to be rented to haul cement, over 4% grades, up to the dam.

The author's statement that hillside plants are desirable,§ does not always hold. It was found desirable for the dam at Hetch Hetchy to provide a mixing and hoisting plant near the foundation, up-stream portion of the dam, and there build a tower, 350 ft. in height, to handle all materials. The reason for this was that any amount of excellent broken fragmentary rock from the granite bluffs was found around the basin within 1 mile of the dam site. It was crushed ½ mile away from the dam, assembled in heaps, and hauled by a separate narrow-gauge railway down to the mixing plant. At first, it was thought that the contractors had made a mistake, and that a high-head plant with a quarry would have been more economical; but the high cost of powder

* City Engr., San Francisco, Calif.

† Received by the Secretary, October 4, 1927.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1725.

§ *Loc. cit.*, p. 1723.

from 1919 to 1923, and the high cost of labor, demonstrated that the contractor's plans, with a crushing plant in the valley and a mixing plant near the dam, were the most economical.

An interesting feature in connection with the dam was that the most attractive and accessible sand in the river bed near the dam and mixing plant was physically undesirable, because for thousands of years a grove of oak trees grew over it. The leaves and drippings from the oaks saturated the sand with tannic acid, which proved fatal to its setting qualities in concrete, and sand was brought from an excellent uncontaminated bed that contained more than 200 000 cu. yd., about 3 miles from the dam up stream.

Mr. Bent's camps are always orderly, sanitary, and well conducted, and all experienced contractors and engineers are now educated to the necessity and desirability of having first-class comfortable camps for the men.

Building contractors will surely fail in dam construction, because it is a business about which they know nothing. The writer knows two instances of builders attempting this kind of construction, with a consequent loss, in one case of more than $100 000 and in the other of more than $200 000. Builders are not trained for successful dam construction.

The most important suggestion Mr. Bent has made is in regard to the handling of water and saving the construction plant from destruction by floods.* It was the writer's duty to compel a contractor to raise a proposed diversion dam more than 5 ft. higher than his contemplated plans in order to deflect a stream of 8 000 sec-ft. during flood periods into a diversion tunnel. Raising the dam was the only thing that saved his plant from destruction and his pocket from a damage loss.

On the whole the author, as well as the Society, deserves credit for bringing the practical man's point of view into the construction of dams, thereby causing engineers to think over many of the problems the contractor has to study before successfully building them.

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* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1726.

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PAPERS AND DISCUSSIONS

THE SEWAGE DISPOSAL PROBLEM OF LOS ANGELES, CALIFORNIA

Discussion*

By A. M. Rawn, Assoc. M. Am. Soc. C. E.

A. M. Rawn,† M. Am. Soc. C. E. (by letter).‡—From Mr. Knowlton's paper, covering the history of the development of the Los Angeles City Metropolitan Sewer System, the writer abstracts two facts of outstanding importance that bear directly on the design and progress of similar works in other parts of California and elsewhere.

It is apparent that there is room for much study regarding the action of sewage when discharged into salt water; especially with reference to the direction and depth of discharge below the water surface and the amount of diffusion into the sea water. The selection of material used in the construction of the ocean section of the outfall plays an important part in determining whether the capacity life will be reached prior to elemental destruction.

The outfall sewer, designed for the City of Los Angeles in 1887, according to Mr. Knowlton, with the ocean end constructed of cast iron and extending 600 ft. into the ocean, provided for a capacity of 19 000 000 gal. per day, which should have provided for disposal into the ocean until about 1910. Nevertheless, it is found to have been superseded twice during its capacity life; once in 1904, with a steel pipe outfall extending 340 ft. farther seaward and, again, in 1908, with a wood stave pipe outfall of a length equal to the steel pipe, but with a different method of discharge into the water at the ocean end.

The wood stave outfall was designed with a capacity of 75 000 000 gal. per day and should have sufficed the needs of the city as an outfall sewer

* This discussion (of the paper by Willis T. Knowlton, M. Am. Soc. C. E., presented at the meeting of the Sanitary Engineering Division, Seattle, Wash., July 15, 1926, and published in August, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Chf. Engr., County Sanitary Dists. of Los Angeles County, Los Angeles, Calif.

‡ Received by the Secretary, October 14, 1927.

until about 1923. It has been superseded twice since construction; once, in 1918, with a wood stave line extending 2 000 ft. seaward and, again, in 1925, with the present outfall which extends about 1 mile seaward, discharging under 60 ft. of water. Thus, eight years prior to reaching capacity, the outfall constructed in 1908 was abandoned for a larger one (which is still in existence); and the latter, in turn, was abandoned for a still longer and larger one.

It would appear from this, which is an outstanding example, and which finds its parallel in many ocean outfalls constructed elsewhere, that sufficient experimental work and study should be conducted to enable the engineer to predetermine, with a fair degree of accuracy, the length of the outfall and other requirements of discharge that will enable him to construct the ocean end of the system with the same degree of permanency as the part on shore.

The second outstanding feature is the policy of the City of Los Angeles in building its sanitary sewer structure for the use of many communities not a part of its municipality and not bonded for its construction and then selling the allocated capacity to such communities, instead of attempting the organization of a large sanitary sewer district in the first place and attempting to persuade the communities for which sewerage facilities were to be provided, to participate in the original cost of construction.

A well constructed outfall sewer is always a merchantible commodity in a growing district if it can be reached by a community which otherwise needs to resort to expensive local treatment, and this is clearly shown by the avidity with which the cities surrounding Los Angeles, and for which capacity has been provided in the North Outfall, are seizing the opportunity to discharge sewage into the North Outfall Sewer System under conditions dictated by the City of Los Angeles.

The writer is an advocate of the sedimentation process, as compared with fine screening, as a means of clearing up the effluent prior to disposal by dilution. Such treatment tends to reduce the dilution necessary, with consequent reduction of ocean-section costs; and, furthermore, the settled material resulting from sedimentation is much more responsive to reduction treatment than it is to screening.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

HENRY STEVENS HAINES, M. Am. Soc. C. E.*

DIED NOVEMBER 3, 1923.

Henry Stevens Haines was a descendant of Deacon Samuel Haines who settled in New Hampshire in 1632. His father, Henry Stevens Haines, had established himself in Wilmington, N. C. His mother was a member of the old Coffin family of Nantucket Island, Mass., and it was during a visit to her mother's home there that Henry Stevens Haines was born, on November 21, 1836.

After the discovery of gold in California his father sent a cargo of building material and partly constructed houses around Cape Horn to San Francisco, Calif., but he himself went by way of the Isthmus of Panama, stopping long enough to accept and complete a contract for cross-ties for the Isthmian road, then under construction. In the California venture he was very successful and was about to return to his home in Wilmington, when he died suddenly of smallpox, leaving his widow with four young sons, of whom Henry Stevens Haines was the eldest.

The boy had a keen mind and a retentive memory. He was taught by two masters in schools, one in Massachusetts and the other in North Carolina. He never went to college, but became a student of men and affairs, a first-class mathematician and geographer, with a good working knowledge of four languages.

Mr. Haines' first connection with railway service was in 1853 or 1854 when he was employed by the Wilmington and Manchester Railroad Company of North Carolina. His next work was with the North Eastern Railroad Company in North Carolina and South Carolina, in the machine shops and as Locomotive Engineer.

When the Civil War began, he was Assistant Engineer on the construction of the Charleston and Savannah Railroad. With his three brothers, he enlisted in the Confederate Army, but soon was returned to railroad construction where he could be of more service, the railroads being most important to the Confederacy. He completed several bridges and other structures in his district and in the course of his work witnessed many battles and was under fire on various occasions. When Charleston, S. C., fell, he was in charge of the transportation of the retreating Southern forces.

After the war, in 1866, Colonel Haines was appointed General Superintendent of the Atlantic and Gulf Railroad which, in common with Southern railroads at that time, was deficient in equipment and was in a bad financial condition. This line extended from Savannah to Thomasville, Ga. From it, however, Colonel Haines developed the Savannah, Florida, and Western Rail-

* Memoir prepared by Charles O. Haines, Esq., Norfolk, Va.

road, which, together with the Charleston and Savannah Railroad, became one of the important systems of the South. He later extended the line through Florida to Tampa, with steamers to Key West and Cuba. With the Alabama, Midland, and Montgomery Railroad, these lines comprised the Plant System, of which Colonel Haines was General Manager and Vice-President from 1882 to 1895.

During these years, Colonel Haines and his wife, Elizabeth Owens, of Charleston, made their home for a time in Charleston and, later, in Savannah. They had six children, of whom only two sons survive him. During his long residence in Savannah, Colonel Haines was active in both City and State affairs whether working in the midst of a yellow fever epidemic or promoting the building of the Georgia School of Technology.

During the last years of his connection with the Plant System, he moved to New York, N. Y. By this time he had long been known as an experienced railroad executive. Largely through his efforts the American Railway Association had been organized, of which he served several terms as President. Colonel Haines was one of the first members of the Southern Society in New York. He was also a member of the Engineers' Club of New York and had served as one of its Vice-Presidents. He and his associates initiated standard time and the standardization of tracks throughout the South.

In 1895, the American Railway Association sent him to England as its representative at the first International Railroad Congress, where his speeches and addresses attracted wide attention. On his return from Europe in the autumn of 1895, Colonel Haines severed his connection with the Plant System to become Commissioner of the Southern States Freight Association, with headquarters in Atlanta, Ga. His wife died in December of that year. He held this office for about two years, when he resigned to travel abroad.

After his return to this country, Colonel Haines was married to Anna Davies, of Detroit, Mich., the daughter of the Bishop of the Protestant Episcopal Diocese of Detroit, who, with their two sons, survives him. For a short time after his return, Colonel Haines was Vice-President of the Atlantic and Danville Railroad Company. While living abroad some years later, he was appointed by the American Government as its representative at the International Railroad Congress held in Berne, Switzerland.

He retired from active business, however, in 1900, and after spending several years in travel, settled with his wife's people in Springfield, Mass., with a summer home at Lenox, Mass., where he died on November 3, 1923.

Colonel Haines' theory of railroad management was based on the delegation of authority as far as possible, but with the proviso that responsibility should rest on the chief operating official. He was of an eminently judicial turn of mind and just in his relations with other men. He was fond of people, always seeking to bring out their best qualities, was never overcritical, and was anxious that all subordinates or business associates should receive every possible acknowledgment of their efficient co-operation or assistance in the successful operation of the railroads entrusted to his care.

His interest in the welfare of the railway employees led to the organization of the Employees' Mutual Relief Association, a benefit and insurance society,

and of the Co-Operative Store. In the latter not only were dividends paid to the stockholders, but there was a division of profits at certain fiscal periods.

At the suggestion of a member of his family, Colonel Haines collected his various addresses and published them, with some re-arrangement, in a book entitled, "American Railway Management". It met with a sufficient demand for him to continue writing on various phases of railway problems, and he wrote continuously until his death. His other works are, "Railway Corporations as Public Servants", "Problems in Railroad Regulation", and "Efficient Railroad Operation".

His was a successful life in its truest sense; his work was well done, and those who were associated with him became his friends and remained so to the end.

Colonel Haines was elected a Member of the American Society of Civil Engineers on November 2, 1887. He served as Director of the Society from 1897 to 1899 and as Vice-President from 1901 to 1902.

CONRADO EUGENIO MARTINEZ Y RENGIFO, M. Am. Soc. C. E.*

Died May 21, 1927

Conrado Eugenio Martinez y Rengifo was born in Havana, Cuba, on April 26, 1881. He was graduated in 1896 from the High School, Santa Clara, Cuba, with the degree of Bachelor of Arts. Soon afterward, he came to the United States and entered Lehigh University, at South Bethlehem, Pa., from which he was graduated in 1901, with the degree of Civil Engineer, the first on the Roll of Honor.

After leaving college, Mr. Martinez obtained a position as Rodman with the Pennsylvania Railroad Company, which position he held until December, 1901.

He then returned to Cuba, and entered the Government Service in the Engineer Department of the City of Havana, starting in January, 1902, as Rodman and working his way up to the position of Superintendent of the Department of Streets and Parks, which position he attained in January, 1903, and held until January, 1907. He was then transferred, as Assistant Engineer, to the Direction General of Public Works. Shortly afterward, he was appointed Engineer in charge of the location and construction of the highway from Cábanas to Bahia Honda, a distance of 20 miles, on the North Coast, which position he held until August, 1907, when he was appointed Chief Engineer of Public Works for the Province of Matanzas, Cuba.

In March, 1909, Mr. Martinez was transferred to Havana as Principal Assistant Engineer of the Havana Sewer and Paving Contract, where he remained until February, 1912. During this period he frequently acted as deputy for the Chief Engineer.

From March, 1912, until August, 1914, he was engaged in important private practice in designing and construction. In August, 1913, he was

* Memoir prepared by Manuel D. Diaz, Esq., Havana, Cuba.

appointed Engineer Member of the National Board of Health which position he filled until shortly before his death. In August, 1914, he was appointed Chief Engineer of the Havana Sewer and Paving Contract and remained on this work until August, 1915, when he was transferred to the Direction General of Public Works as Chief Engineer in charge of examination of projects and inspection of new construction work for the Department of Public Works. In November, 1916, he was appointed Chief Engineer in charge of the Sewerage and Paving Contract for the City of Cienfuegos, Cuba, and in June, 1917, he was made Engineer in charge of the construction of water-works for the City of Marianao, Cuba.

In June, 1918, Mr. Martinez engaged once more in private practice as Consulting Engineer, in partnership with his brother, Rolando A. Martinez, Assoc. M. Am. Soc. C. E. In that capacity he undertook varied and important engineering enterprises, such as bridge and wharf construction; the design and construction of water-works and sewers; extensive land sub-divisions; surveys; appraisals; design of molasses pumping plants, etc., until his death.

On November 22, 1924, Mr. Martinez received the degree of Doctor of Laws, from the University of Havana after having taken the full course prescribed for the degree. Subsequently, while engaged in engineering work, he found time for the successful practice of his new profession.

Although financially independent, his love for the profession originally chosen by him, made him an indefatigable and enthusiastic toiler. A lover of truth, he spoke it, and wrote it irrespective of its bitterness. His blunt honesty with his modesty and great worth made him many friends to whom his untimely death was as severe a shock as it was to his family and to those closely associated to him socially and professionally.

Mr. Martinez is survived by his widow, and two daughters, Clara and Blanca, to whom he leaves an example of faithfulness, truth, and integrity.

He was a member of the Union Club, of the Country Club of Havana, of the Association of Members of American National Engineering Societies in Cuba, and of the Tau Beta Pi Fraternity.

Mr. Martinez was elected an Associate Member of the American Society of Civil Engineers on October 7, 1908, and a Member on September 9, 1919.

THOMAS HOGGAN MATHER, M. Am. Soc. C. E.*

Died March 23, 1927.

Thomas Hoggan Mather, the son of William and Isabella Hoggan Mather, was born near Glasgow, Scotland, on May 2, 1860. His father was an engineer and encouraged his son to follow the same profession.

Mr. Mather's early engineering training was received as an articled pupil with Kyle, Dennison, and Frew, Civil Engineers of Glasgow, and as a student in special classes at Glasgow University.

Upon the completion of his university study in 1879, he was appointed Surveyor of Roads, Cadder Parish, Lanarkshire, Scotland.

* Memoir prepared by Glenn D. Holmes, M. Am. Soc. C. E.

In 1882, Mr. Mather removed to Canada and for the ensuing nine years he was engaged in a variety of work in that country and in the United States. He served consecutively as Transitman for the Canadian Pacific Railway; Superintendent of the Las Vegas, N. Mex., Water System; Office Engineer and Chief of Location Party for the Canadian Pacific Railway; Chief of Location Party for the Quebec Central Railway and for the Témiscouata Railway; Chief Engineer on Reconnaissance for the Gaspé Short Line; District Engineer on Location and Construction for the Calgary and Edmonton Railway; and District Engineer for the Norfolk and Western Railroad.

In 1892, he removed to Syracuse, N. Y. With the exception of the four years from 1918 to 1921, he resided there continuously until his death.

A period of private practice in partnership with Henry C. Allen, M. Am. Soc. C. E., was followed by twenty years' service as Chief Engineer of construction and maintenance for high-speed electric railways. During 1915, Mr. Mather served as Commissioner of Public Works for the City of Syracuse. Two years of private practice were succeeded by more than three years of Government service as Supervising Engineer of Construction at Harrisburg, Pa., and as Superintendent of Utilities at the Quartermaster Corps Depot in the same city.

After a brief interval when he was engaged in private practice in Brooklyn, N. Y., he was appointed City Engineer for the City of Syracuse in January, 1922, which position he held for four years. At the time of his death, Mr. Mather was serving as Chief Engineer of the Syracuse Grade Crossing Commission, having been appointed to that position in January, 1926.

He is survived by his widow, Mrs. Isabella Layfield Mather, and by two daughters, Mrs. Harry H. Motheral, of Brooklyn, N. Y., and Mrs. John C. Adams, of Jersey City, N. J.

Mr. Mather was a member of the Syracuse Chamber of Commerce, Technology Club, and Citizens' Club of Syracuse. He was also affiliated with Central City Lodge, F. and A. M., of which he was a Past Master, and of Central City Commandery No. 25.

As he was endowed with good humor and possessed of a fund of good stories effectively told, he contributed enjoyment to any festivity which his presence graced. For recreation, Mr. Mather sought the woods, at home and abroad, and the wild plant life—particularly the ferns—to be found there. His personal friends of whom there were many, found attraction and pleasure in his conversation and company.

The greater part of his life was spent in railroad work, the branch of the profession which he most enjoyed. Those associated with him acquired a high esteem for his professional ability and his attractive personal qualities. The Grade Crossing Commission with which he was engaged at the time of his death, recorded a tribute to his exceptional skill, his impartial attitude to the solution of its many problems, his unlimited application to his work, and the serious and sincere devotion to his duty. The City of Syracuse has lost a very able, conscientious, and valuable public servant.

Mr. Mather was elected a Member of the American Society of Civil Engineers on October 1, 1902.

CHARLES JULIUS POETSCH, M. Am. Soc. C. E.*

Died October 7, 1926.

Charles Julius Poetsch was born in Wriezen, near Berlin, Germany, on July 17, 1850. He was educated in the public schools and the German Real Schule (a college), and came to Milwaukee, Wis., in 1871.

Mr. Poetsch began his engineering work in the United States as a Draftsman and, later, was appointed Assistant Engineer on the survey and construction of the Milwaukee and Northern Railway, having been with this Company from 1870 to 1874. In 1875, he was Assistant Engineer for the Green Bay and Western Railway Company, all this work having been done in the State of Wisconsin. He then became Resident Engineer for the Chicago, Milwaukee and St. Paul Railway Company, at Minneapolis, Minn., which position he held from 1875 to 1878.

In 1878, he returned to Milwaukee, where he was appointed a Division Engineer in the City Engineer's Office, also acting as Assistant City Engineer. This work consisted of giving lines and grades for various street construction, water mains and sewers, bridge construction, and other miscellaneous municipal improvements. He remained in this position until 1899 when he was appointed City Engineer.

In this capacity, Mr. Poetsch was also *ex officio* President of the Board of Public Works, and his guidance in the policy of local improvements enabled the City to undertake many notable projects. The construction of the Grand Avenue Bridge, the first bascule bridge erected in Milwaukee, was directed by Mr. Poetsch. Since then all other bridges erected by the City have been of this type. Two long viaducts of steel construction, spanning the lowlands of the Menomonee River Valley, were also designed and constructed under his direction.

A flushing tunnel, a pumping station, and an intake from Lake Michigan, delivering water to the upper reaches of the Kinnickinnic River to prevent stagnation of the river waters during the low-flow periods of the summer, was also built by Mr. Poetsch.

The water-works of the city were also directly in his charge, as Superintendent. In that capacity he directed the purchase of pumping equipment, the first enlargement of the present North Point Pumping Station on the shore of Lake Michigan, the inland high-service pumping station, and the extensions of large feeder mains and the pumping station at West Allis, Wis., a suburb of Milwaukee. The first method for treating the water supply with chlorine, using hypochlorite dosing, was designed by Mr. Poetsch.

In all his works, he was ever thoughtful of economy in construction, and the various local improvements were made at the greatest saving to the citizens.

In 1911, Mr. Poetsch severed his connection with public work and engaged in private practice. He established the firm of Poetsch and Geiger, Con-

* Memoir prepared by C. S. Gruetzmacher, Assoc. M. Am. Soc. C. E., and H. P. Bohman, Supt., Water Dept., Milwaukee, Wis.

sulting Engineers on municipal work of all kinds. Ill health in 1916, and subsequent thereto, caused the abandonment of his private practice and forced his retirement from active engineering work. He was President of the Cummings Boulevard Land Company and a Director in the Home Savings Bank of Milwaukee, and he devoted his time to these and other business activities in which he was engaged.

During the later years of his life, Mr. Poetsch always found time to revisit the scenes of his former activities in the City Engineer's Department, lending able advice. He was a frequent visitor at the meetings of the local engineering societies and the Old Settlers' Club of which he was a member. He was also a member of the Wisconsin Lodge No. 1, Knights of Pythias. During his long services in public office, he gained a vast acquaintance by whom he will be greatly missed.

He was married in 1873 to Minnie E. Rausch. They had one daughter, now Mrs. Daisy L. Laming, of Tonganoxie, Kans., who, with her mother, survives him.

Mr. Poetsch was elected a Junior of the American Society of Civil Engineers on May 4, 1881, and a Member on May 2, 1883.

JOHN DUBUIS, Assoc. M. Am. Soc. C. E.*

Died May 11, 1927.

John Dubuis was born on October 8, 1884, in the Tyrol, Austria. His mother died soon after his birth and his father, an American artist, working and studying in Europe, died three years later while still abroad. The 3-year old son was brought back to the United States, and, as a boy, lived in Kentucky and South Carolina. He was graduated from Presbyterian College of South Carolina in 1905, and received the degree of Civil Engineer from Cornell University in 1909.

Mr. Dubuis moved to Oregon in 1909, and for the next eighteen years he was associated with many of the more important engineering developments in that State. He was engaged in investigation of irrigation projects in Warner Valley and at Grant's Pass, and in the construction of the Hood River Hydro-Electric Plant, of the Pacific Power and Light Company, and the irrigation system of the Walker Basin Irrigation Company, near Lapine.

As Chief Engineer, he supervised the construction of the Gold Hill Irrigation District System. He was also Consulting Engineer for the North Canal Company from 1921 to 1924, and for several smaller projects. He served one and one-half years as Instructor in Hydraulics at the Oregon Agricultural College, and for three years as Engineering Inspector for the Desert Land Board of the State of Oregon. Mr. Dubuis' most important work was no doubt the water supply system for the City of Bend, Ore., involving a 13-mile pipe line, intake dam, reservoirs, etc., recently completed. His

* Memoir prepared by Fred F. Henshaw, M. Am. Soc. C. E.

death occurred at Spokane, Wash., as the result of infection following an operation for appendicitis.

He was married in 1913 to Marion K. Curtis, of Ithaca, N. Y. He is survived by Mrs. Dubuis and by two daughters, Marion and Jeanne.

Mr. Dubuis was very active in Masonry, having been a member of the Portland Lodge No. 55, Pilgrim Commandery of Knights Templars, of Bend, Al Kader Temple of the Mystic Shrine, and Bend Chapter No. 109 of the Eastern Star. He was especially active in the Pilgrim Chapter, De Molay, having been a member of the Advisory Board, and in the Boy Scouts, having served as Scoutmaster.

He was beset with the necessity common to most civil engineers of frequent removals from place to place; nevertheless, he was able to make a host of friends all over the State of Oregon. Those who knew him best will probably agree that his outstanding personal characteristic was his never-failing cheerfulness, even in the face of discouragement and adversity.

Mr. Dubuis was elected a Junior of the American Society of Civil Engineers on October 1, 1912, and an Associate Member on January 6, 1915.

JESSE ALBERT CURREY, Affiliate, Am. Soc. C. E.*

Died June 23, 1927

Jesse Albert Currey, the son of William B. and Anna Mary (Cloud) Currey, was born at Philadelphia, Pa., on August 2, 1873.

His first employment of a business nature probably moulded his career to a large degree for at the age of nineteen, he entered newspaper work in Philadelphia and continued in that field in that city and in Wilmington, Del., for ten years.

In 1902, he entered the engineering field by appointment as Superintendent of drilling operations for the Eastern Texas Oil and Mineral Company in the vicinity of Hemphill, Tex.

Mr. Currey devoted the next two years to work for the Baltimore Construction Company which had under way the construction of piers at Locust Point for the Baltimore and Ohio Railroad Company and the North German Lloyd Steamship Company, as well as improvements to docking facilities in Baltimore, Md., Harbor. During his connection with that organization, Mr. Currey held the position of Paymaster, Superintendent, and, finally, that of General Manager.

Traveling next claimed his attention and, beginning in 1904, he toured extensively throughout the United States. He finally located at Portland, Ore., where he first became engaged in the real estate business. In 1906, he was made Pacific Northwest Manager for the Truscon Steel Company, which position he held at the time of his death.

* Memoir prepared by J. C. Stevens, M. Am. Soc. C. E.

His connection with the Truscon Steel Company brought him in close contact with the major building operations in his territory, and he aided greatly in the design of large viaducts, hospitals, power stations, as well as bridges and buildings for various railroads. When he found his Company had no effective means of cutting metal lath, Mr. Currey invented a machine for this purpose.

He was married, in Philadelphia, on April 26, 1905, to Frances Whiteley, who survives him.

Although Mr. Currey had been affiliated with various engineering enterprises during the last twenty-five years of his life, he was perhaps better known throughout the United States and foreign countries for his activities in the culture of roses.

He was a member of the Royal Horticultural Society and the National Rose Society, of England, and a Director of the American Rose Society at the time of his death. His life work in the field of his avocation brought him honor and fame.

He was also a member of the Portland Rotary Club and the Portland Chamber of Commerce. His activities in both organizations were earnest and painstaking. He was always ready to undertake any task great or small that had to do with the betterment and upbuilding of the community in which he lived.

Mr. Currey was elected an Affiliate of the American Society of Civil Engineers on February 1, 1910. He was very active in local Society matters, seldom missed a meeting of the Portland Section, and served with great credit on several important committees.

MOTT TITUS CARROLL, Jun. Am. Soc. C. E.*

DIED AUGUST 5, 1927.

Mott Titus Carroll, the son of Edward T. and Virginia (Titus) Carroll, was born at El Dorado, Kans., on July 31, 1897. His father was a native of New York, N. Y., and his mother of Racine, Wis.

Mr. Carroll received his technical education at the Kansas State Agricultural College, at Manhattan, Kans., from which he was graduated in June, 1926, with the degree of Bachelor of Science in Civil Engineering. He was a brilliant scholar and was awarded the Kansas State Section Prize of Junior Membership in the Society.

During his summer vacations, while at college, he was engaged as Rodman and Draftsman with the Butler County Highway Department, at El Dorado. After his graduation, Mr. Carroll was appointed Resident Engineer with the Kansas Highway Commission, in charge of the construction of Federal Aid Project No. 333, near Sharon, Kans. After completing this work, he accepted a position as Bridge Designer and Draftsman with the Butler County Highway Department, at El Dorado. This position he held at the time of his death.

* Memoir prepared by E. S. Elcock, Assoc. M. Am. Soc. C. E.

Mr. Carroll was just entering on his professional career, and his energy, perseverance, and keen mind gave promise of a successful future. His kindly personality will be greatly missed by his friends and associates.

On July 14, 1927, he underwent an operation for appendicitis, resulting in complications which caused his death on August 5. He is survived by his widow, two small children, his father, and a sister, Mrs. B. D. Sisson.

He was a member of Phi Delta Theta Fraternity and Wichita Lodge No. 99, A. F. and A. M.

Mr. Carroll was elected a Junior of the American Society of Civil Engineers on January 17, 1927.